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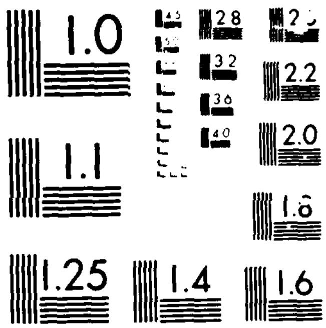

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An Investigation Conducted By
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NCEL
Contract Report

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Engineering Command

PRESTRESSED CONCRETE FENDER PILES - ANALYSIS AND FINAL TEST PILE DETAILS

ABSTRACT Using prior test results this study refined an analytical model investigating various alternatives to optimize concrete pile design for Navy piers. Parameters evaluated were: size, prestressing force, load application, ductility, placement of reinforcement, and concrete strength. It was demonstrated that prestressed concrete piles will outperform steel and timber piles in a pile-for-pile energy comparison and are more cost-effective. Recommendations are made for a final test program.

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SECTION 1
SUMMARY

The Department of the Navy, through the Naval Civil Engineering Laboratory (NCEL) at Port Hueneme, California, has initiated a program to develop prestressed concrete fender piles for use at Navy piers in a wide range of fendering applications. This report completes the analysis model and final test pile detailing portion of the multiphase analysis, testing, and design effort [1.1].

The preliminary test pile program successfully demonstrated the concept of prestressed concrete fender piles. The test piles surpassed predictions of the Phase I report and demonstrated good cyclic behavior. The analytical model and pile design were further refined in this phase.

Major tasks in this phase were to refine the analytical pile model, investigate various alternatives and optimize the pile designs, and provide recommendations for the final test pile program including drawings and specifications. As a part of this work, a computer program was developed for use in future phases of the program. Other tasks in this phase were to perform life-cycle cost estimates for various alternatives including steel and timber piles for comparison, investigate rub strip materials and connection details, and prepare preliminary cost estimates for foam fender system concepts employing precast prestressed concrete piles.

A revised analytical model and computer program were developed which accurately and conservatively predict the behavior of prestressed concrete fender piles. The revisions were based on the results of the preliminary test pile program. The materials and analysis procedures recommended in this study essentially represent standard practice in the prestressed concrete industry.

Various pile alternatives and parameters were investigated including pile size, prestressing, load application, ductility, reinforcement placement, and concrete strength. The pile configuration recommended as a result of this study is very similar to the Phase I pile except that the reinforcement and its placement have been optimized to achieve a specific energy level of 30 ft-kip at ultimate.

Corrosion resistance of concrete piles was reviewed. Prestressed concrete piling has an excellent service record. A review of the literature and interviews with experts on corrosion and materials were conducted; among the items investigated were quality of concrete, quality of fabrication, epoxy coating of reinforcement, cathodic protection, and coatings. The most effective corrosion protection is the use of high-quality, low-permeability concrete. The effects of multiple cracks should be investigated. If multiple cracks are a problem, then one alternative to consider would be epoxy-coated strand. Coatings on prestressed fender piles would normally not be required.

Six rubbing strip alternatives were developed. The rub strip provides a smooth surface between the pile and ship. Where a log camel is used, no rub strips would be provided. Three different materials and attachment details were investigated. The least costly and simplest alternative would be to use ultra high molecular weight (UHMW) plastic bolted directly to the pile. The optimum length for the rubbing strip is suggested as 14 ft.

A survey of precast concrete fabricators was conducted to determine regional capabilities to produce prestressed concrete fender piles. A questionnaire was developed relating to pile size, concrete strength, concrete additives, and epoxy-coated strand. The 17 responses received were generally favorable to meeting the fabrication requirements being considered by this study; however, some concern was expressed about epoxy-coated strand and steel-fiber-reinforced concrete.

It was demonstrated that properly designed prestressed concrete piles will outperform both steel and timber piles in a pile-for-pile energy comparison.

Steel piles are much stiffer than concrete piles; hence, for an equal energy level, two steel piles were required which produced much higher reaction forces. Timber piles are more flexible (softer) than concrete piles; yet, to achieve the same energy level, four timber piles were required.

The energy costs for equivalent piles (equal energy) and comparable fender systems for prestressed concrete, steel, and timber piles were calculated. The prestressed concrete piles, both individually and incorporated in a system, are more cost-effective than the other two materials.

When using precast concrete piles in a foam fender system where the reaction to the pile is more likely to occur near the top of the pile, it was determined that the "soft" pile would perform as well, if not better, than a "stiff" pile. The major difference between soft and stiff concrete piles, as used in this study, was the level of initial prestress in the strands; i.e., the soft pile has a low level of initial prestress while the stiff pile has a much higher level of initial prestress. The ultimate moment capacity and reaction were the same for both piles but the deflection (and hence energy absorption) was different. Thus, to gain the most use out of a precast concrete pile in a foam fender system and also guard against accidental impacts lower on the pile, a soft pile would be the optimum solution, as more total energy can be absorbed.

As a result of this phase, recommendations are presented for fabrication of piles to be used in the final test program to be conducted by NCEL. The primary goals were to expand on the cyclic behavior of the piles and to improve pile behavior in the postelastic range. A total of 16 piles were recommended for further testing. A pile drawing and specifications we e prepared for test pile fabrication.

SECTION 2
INTRODUCTION

The major effort in this phase was directed toward refining the analytical model for the prestressed concrete pile used in Phase I, optimizing the pile for design and fabrication, investigating the application of the pile in a foam fender system, and preparing final test pile detailing to be based on the most promising pile design. The basis for this work was the Phase I report prepared by ABAM [2.1] and the preliminary pile testing performed by NCEL [2.2].

2.1 PRELIMINARY TEST RESULTS AND ANALYSIS MODEL

After reviewing the results of the preliminary tests, it was determined that the original analytical model underestimated the ultimate capacity of the test piles. An investigation of analytical model assumptions was undertaken. The most significant assumption was the type of concrete stress block to use. A trapezoidal stress block was investigated and adopted for use in this phase because it approximated the test pile results very closely.

2.2 PILE PERFORMANCE CRITERIA

In order to optimize the pile, it was necessary to establish performance criteria. Two criteria are suggested based on concrete behavior and fender system design:

- Pile performance criteria for the concrete were developed from the preliminary test pile results and principles of ultimate strength design for concrete. The cover of the test pile spalled abruptly at concrete strains slightly in excess of 0.003 in./in.

Although additional energy absorption beyond this point was possible, the pile was no longer usable. A maximum concrete strain of 0.003 in./in. was therefore adopted for use with the design program.

- In current practice, most concrete structures are designed for ultimate load-carrying capacity and also checked to see that they do not exceed serviceability limits. However, steel and timber fender pile systems are typically designed for working level loads. Load factors are applied to obtain an ultimate load if ultimate design is used. For this study, the Navy berthing loads were assumed to be working loads. Therefore, as most of our work has been focused on the ultimate capacity of the pile, it was necessary to develop a two-stage criteria, working load behavior and ultimate load behavior, to establish a common baseline for comparison with steel and timber fender piles.

2.3 PILE OPTIMIZATION

An optimization study was undertaken to minimize pile size and reinforcement required. An ultimate pile impact energy of 30 ft-kip was chosen to remain consistent with the work in Phase I. A review of DM 25.1 for various Navy ships indicates that this is a reasonable upper bound of pile energy for most Navy vessels except carriers. Thus, the majority of prestressed concrete fender piles would be within the energy levels investigated and tested. It was rapidly determined that an 18-in.-square pile would be required to achieve this energy level. See Section 4.3 for further details. Based on the optimization study, the optimum pile to achieve the 30-ft-kip energy level was determined to be very similar to the MK-5 test pile from the preliminary testing program. The primary difference was the reduction in strands from 20 to 16. See Figure 2.1 for the cross section of the optimum pile. Smaller piles may be appropriate for piers with lower impact energy requirements.

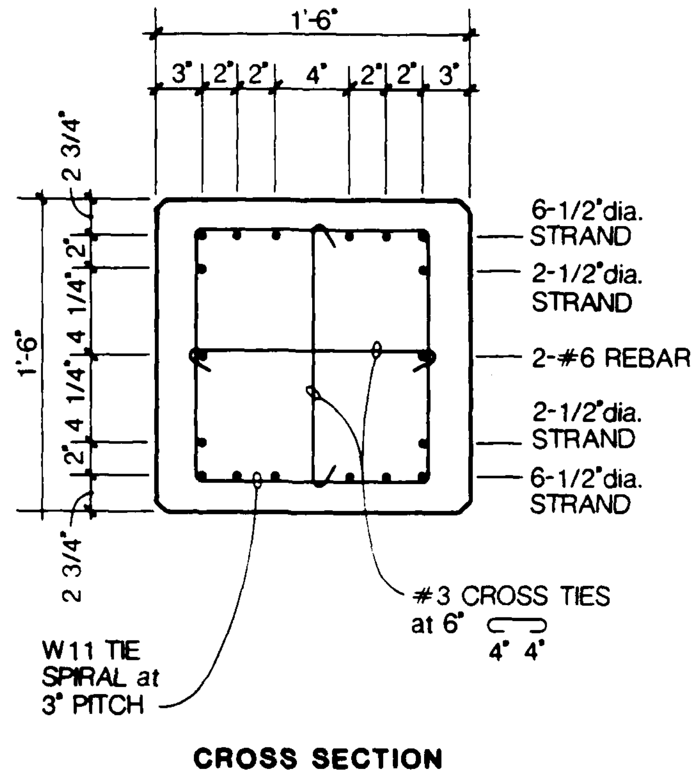


Figure 2.1. Optimum Pile Cross Section
for 30-Ft-Kip Energy Level

Lightweight concrete was considered in Phase I of the study and was tested in the preliminary testing program by NCEL. Based on the fact that it did not show any significant structural benefits [2.2], it was not considered in this phase of the work.

2.4 COST COMPARISONS

The costs of prestressed concrete fender piles were compared with more conventional systems made from steel and timber. On a pile-for-pile basis, prestressed concrete piles can absorb significantly more energy than either steel or timber piles. On a system basis, a large energy-absorbing rubber fender is required in a steel system to absorb the energy. The timber system, on the other hand, is a very good energy absorber, but the total energy that a timber pile can absorb is severely limited. With these constraints, the prestressed concrete fender system was determined to be the most cost-effective system.

SECTION 3
ANALYTICAL MODEL

3.1 PRELIMINARY TEST RESULTS VERSUS PHASE I ANALYSIS

In Phase I of the prestressed concrete fender pile study, ABAM designed test pile specimens to be built and tested at NCEL. The eight piles were all 18 in. square with a test span length of 58 ft 1 in. All the piles were loaded at 15 ft 0 in. from an end support. The baseline case pile was MK-5, with a cross section with 20 strands. It had 20 1/2-in.-diameter strands arranged in a rectangular pattern. The effective prestress in the strands was 60 ksi. Of the eight piles tested, the baseline case was predicted to behave the most efficiently for the amount of prestressing steel used. This was confirmed by the testing at NCEL. The first goal in Phase IV was to try and match the behavior of the test piles using the analytical model developed in Phase I.

The analytical model takes into account the inelastic properties of these materials: concrete, prestressing strand, and reinforcing steel. It is based on the assumption that plane sections remain plane and that deformations are small. For a given cross section, a moment-curvature analysis was performed using strain compatibility between the concrete and steel. The ultimate moment capacity of the section is based on a maximum concrete strain of 0.003 in./in. The energy per unit length for the cross section is based on the area under the moment-curvature curve for the moment applied over the unit length. The summation of the unit energy along the length of the pile gives the total energy that the pile can absorb. The calculation of the pile deflection is based on the energy-to-load

relationship. For more detailed information on the analysis theory used, see the Phase I report [2.1].

A computer run was made for the baseline test pile, MK-5, using the specified design material properties (for example, normal weight concrete, f'_c = 8000 psi) from the Phase I report and the actual test span length of 58 ft 1 in. Figure 3.1 shows how the analytical model results compared to the actual preliminary test results. The model's ultimate load, deflection, and energy were significantly less than the actual test pile. These differences made it necessary to adjust the analytical model and revise the computer program.

The calculation procedure in the Phase I analysis utilized both hand calculations and two separate computer programs. A combined computer program was written in BASIC to do the complete analysis in Phase IV. The name of the program is FENDER.BAS (referred to as FENDER in the remainder of this report). The constraints in the program include a constant rectangular cross section along the length of the pile and single-point loading on the pile. The end conditions are assumed pinned at the top and bottom of the pile. The pile is only driven far enough in the soil to provide lateral support but not to develop fixity. The computer program will be explained further in Section 3.3.

3.2 MODIFICATIONS TO ANALYTICAL MODEL

In order to make the analytical model results more closely match the actual test results, the first items that were checked were the actual material properties of the steel and concrete. The NCEL report on preliminary tests included mill certificates for both the prestressing strand and the reinforcing bars. The actual yield strength of the reinforcing bars was 81.3 ksi versus the minimum specified strength of 60.0 ksi. There are two different types of strand: stress relieved and low relaxation. Stress-relieved (SR) strand has a specified yield stress of 0.85 x f_{pu} while low

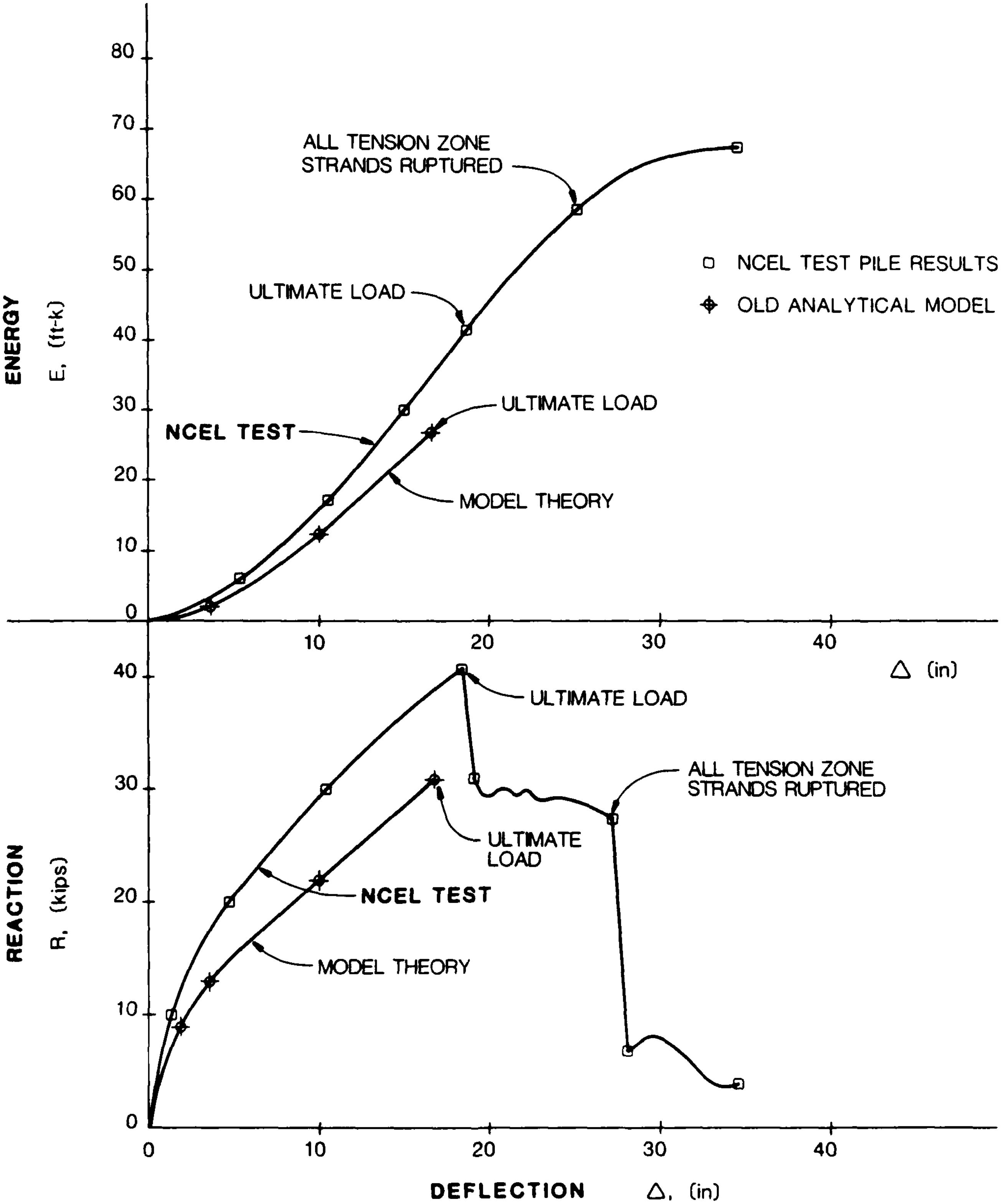


Figure 3.1. Comparison of NCEL Test Results
with Phase I Analysis Model Theory

relaxation (LR) strand has a specified yield of 0.90 x f_{pu}. The model is set up to handle either type of strand. All the tests and analyses are based on LR. The prestressing strand had an actual tensile strength of 280 ksi versus the specified tensile strength of 270 ksi. The actual modulus of elasticity for both the mild and prestressing steel was very close to the specified values.

Trial computer runs showed that the predicted and preliminary test results for test pile MK-5 were not in agreement using the material properties stated in the preliminary test pile report [2.2]. After reviewing the preliminary test result summaries for piles MK-5 and MK-6, it was discovered that the reported concrete strength from the test cylinders for MK-5 was significantly lower than for MK-6 (and lower than the minimum specified design strength), although the piles behaved similarly. Therefore, at the start of this phase of work, several core samples were taken from test pile MK-5 and tested in compression. The average compressive strength, f'_c, of the concrete cores was 9970 psi. This is much higher than that reported from the test (f'_c = 7740 psi) [2.2]. The concrete mix used a high early strength cement and thus should not gain a significant amount of additional strength after the testing; therefore, it was concluded that the core samples were more representative of the actual strength of the pile, at the time it was tested, than the cylinder breaks. Per ACI 318-83, Section 4.7.4 [3.1], these cores are representative of the minimum strength of the parent concrete. The actual strength may be up to 18% greater (1.0/0.85).

The modulus of elasticity of the concrete was originally calculated using the ACI code equation [3.1]. As a result of the testing, NCEL recommended that the Morales equation be used for high-strength concrete [3.2]. Juan Pastor of Cornell University, in his report on high-strength concrete entitled "Behavior of High Strength Concrete Beams" [3.3], also recommends that the Morales equation be used. By using the test results of load-deflection gauges at two locations on test pile MK-5, the modulus of elasticity was calculated at the

point of initial cracking. Again, the results closely matched the Morales equation.

Design codes recognize the maximum concrete strain to be 0.003 in./in. The results in the NCEL test for MK-5 measured an ultimate strain of 0.00321 in./in. For the purpose of matching the test results with the analytical model, the higher value was used. See Section 3.3.1 for recommended design assumptions.

As the model was being refined and compared with the test results, the first portion of the model load-deflection curve was consistently below that of the test pile. This was attributed to the concrete tension capacity, especially under monotonic loadings (not cyclic loadings). This makes the pile stiffer initially, but the effect drops off as the crack depth propagates up the section and approaches the neutral axis. For the analysis model, a maximum tensile stress in the concrete of $f_t = 10\sqrt{f'_c}$ was used. This is also based on the recommendation of Pastor [3.3].

Intermediate computer runs were made reflecting the previously mentioned updates. Each time, the results were compared with those of the test. The ultimate moment capacity of the section for the analytical pile continued to be less than that of the test pile; hence, the total pile energy was also less. Up to this point in our analysis the Portland Cement Association (PCA) stress-strain curve for concrete was used. It is based on a parabolic stress block up to a maximum stress of 0.85 f'_c, which was developed for lower strength concrete. It apparently does not correctly model high-strength concrete behavior. Pastor addresses this topic in his report, both analytically and with test results. In it, he uses a trapezoidal stress block, as shown in Figure 3.2. With the trapezoidal stress block model, the concrete compressive stress starts out triangular in shape and increases to a maximum stress of $\alpha f'_c$ at a strain of $\alpha f'_c/E_c$ (ε_{pz}) in the concrete. For strains greater than ε_{pz}, concrete stress remains constant at $\alpha f'_c$ where α has been

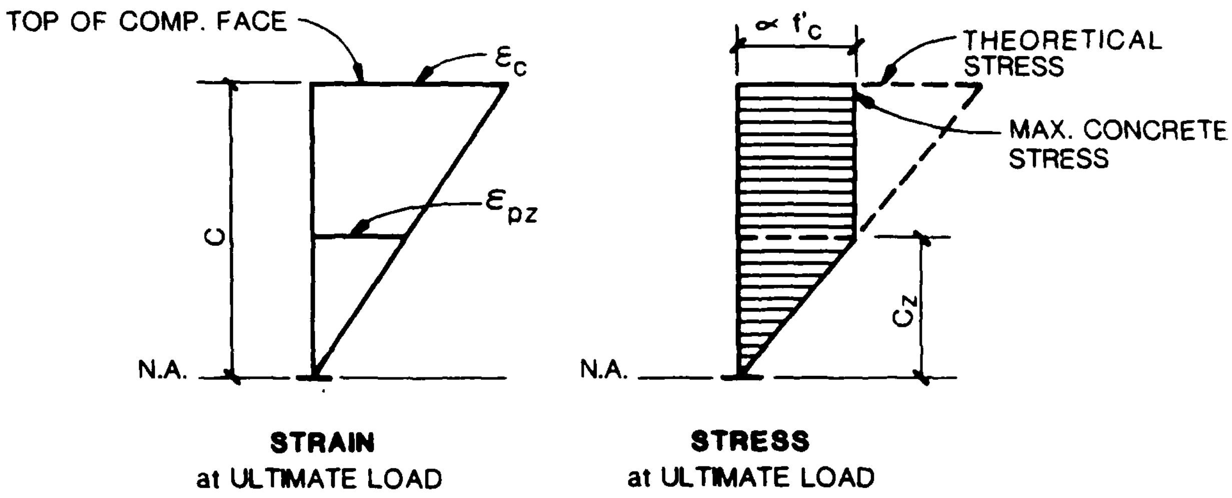


Figure 3.2. Trapezoidal Stress Block

calculated as 1.0 to match the test results. Figure 3.3 shows different stages of concrete stress up to the limiting ultimate strain. The effects of using the trapezoidal stress block and using a maximum concrete stress of f'_c, very closely approximate the test results. See Figure 3.4 for the analytical results plotted in relation to the NCEL test results for test pile MK-5.

To more accurately predict the pile response, the trapezoidal stress block is used, rather than the ACI code equation for the concrete stress in the analytical model. The ACI equation was intended to model the internal forces only at the instant the section reaches its ultimate strength. It is not a good representation of the concrete compression force at lower load levels. On the other hand, the trapezoidal stress block model represents a stress block which varies under different load levels from triangular at low stresses to trapezoidal at ultimate strength. Because the total energy of the pile is based on the summation of the unit energies along the pile which are all at different stress levels (see Figure 3.3), the trapezoidal stress block predicts a more realistic behavior.

In addition, the revised analytical model described above was used to compare the preliminary test results for test piles MK-3 and MK-7. As shown in Figures 3.5 and 3.6, the results closely match the test results.

The revised analytical model has been applied to all three test pile configurations with the exception of the lightweight concrete pile (MK-4) and demonstrated good correlation with the test results. Lightweight concrete did not show any significant structural benefits [2.2], and hence was not considered in this phase of the work.

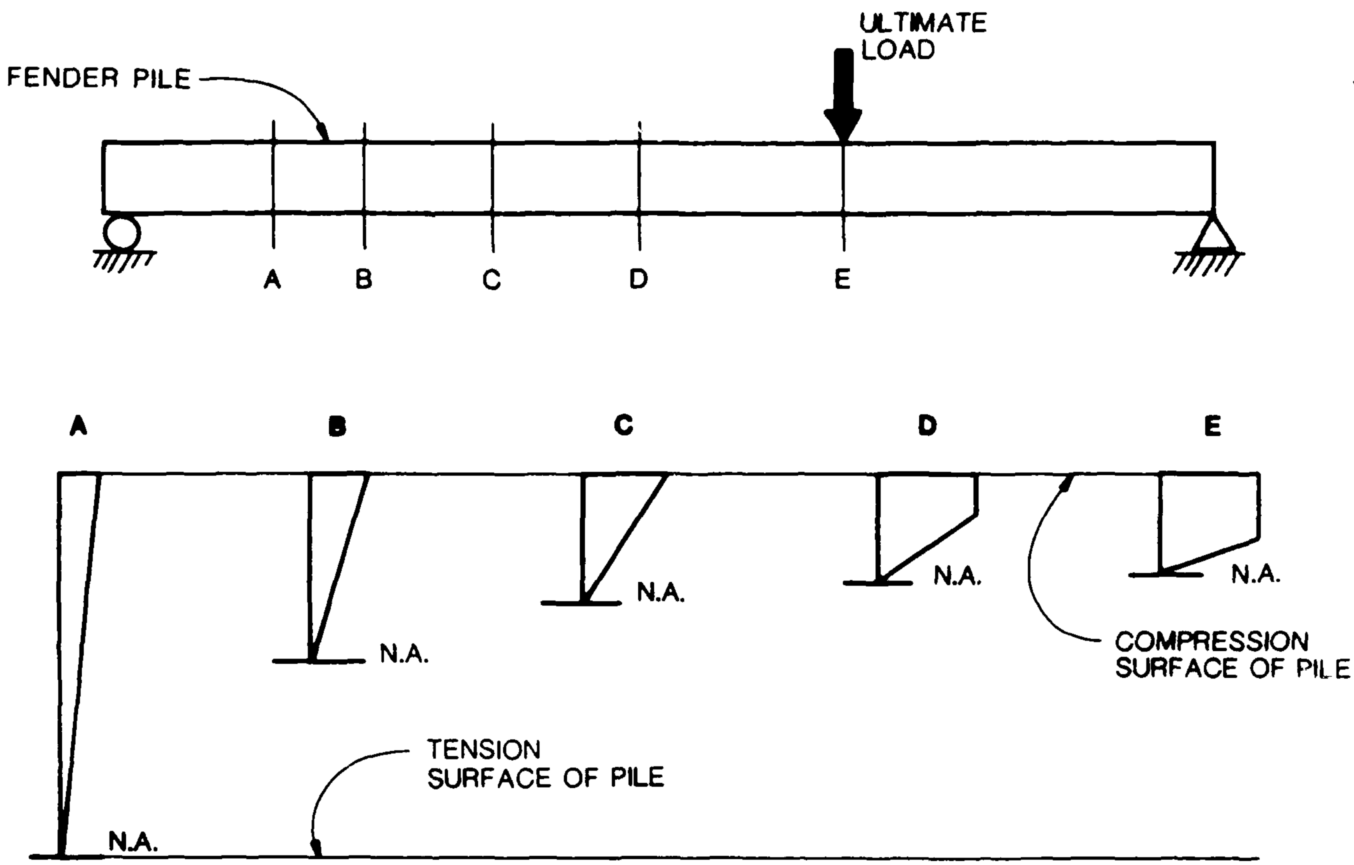


Figure 3.3. Different Stress Blocks Along the Beam

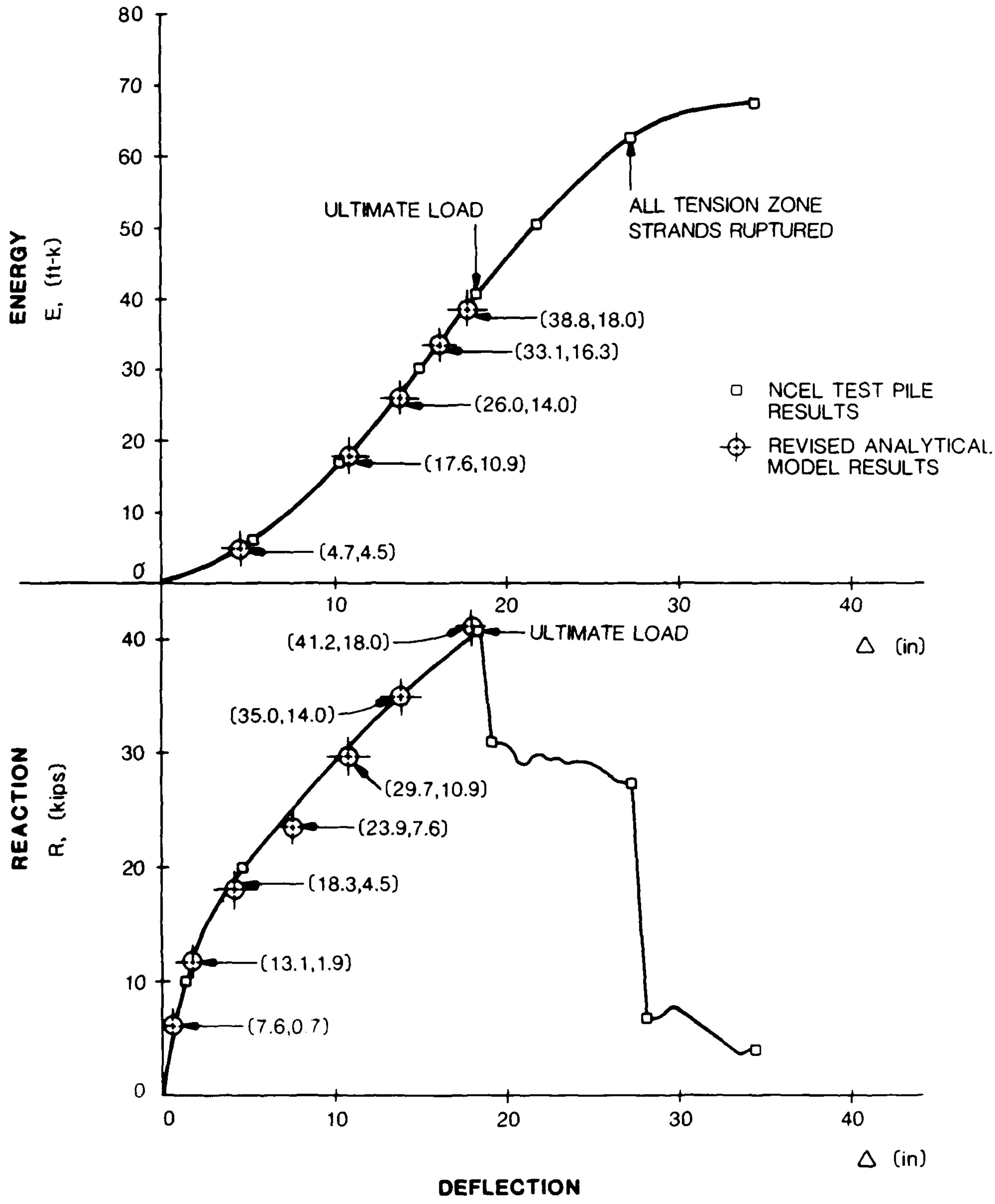


Figure 3.4. Comparison of NCEL Test Results with Test Pile MK-5

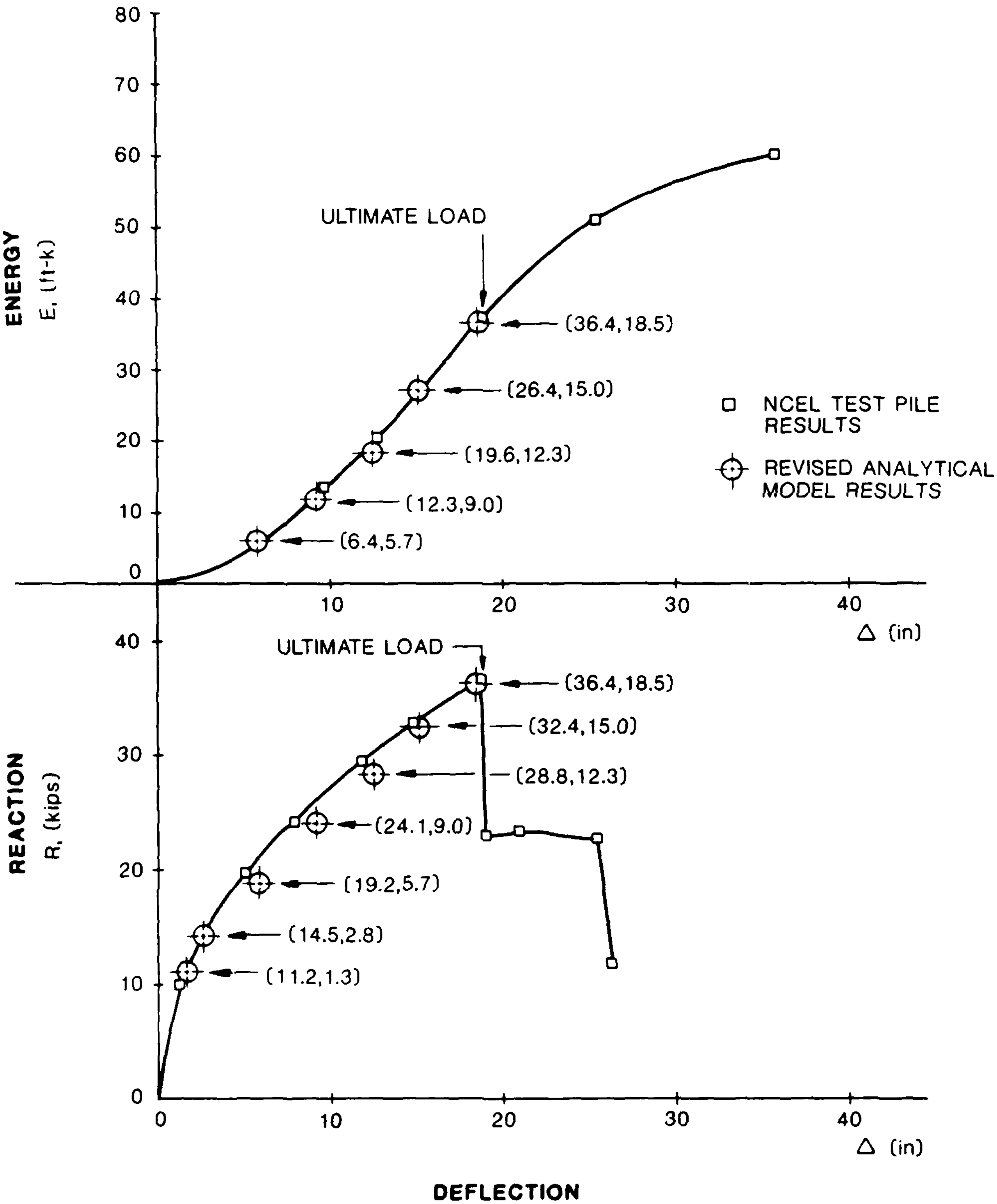


Figure 3.5. Comparison of NCEL Test Results with Test Pile MK-3

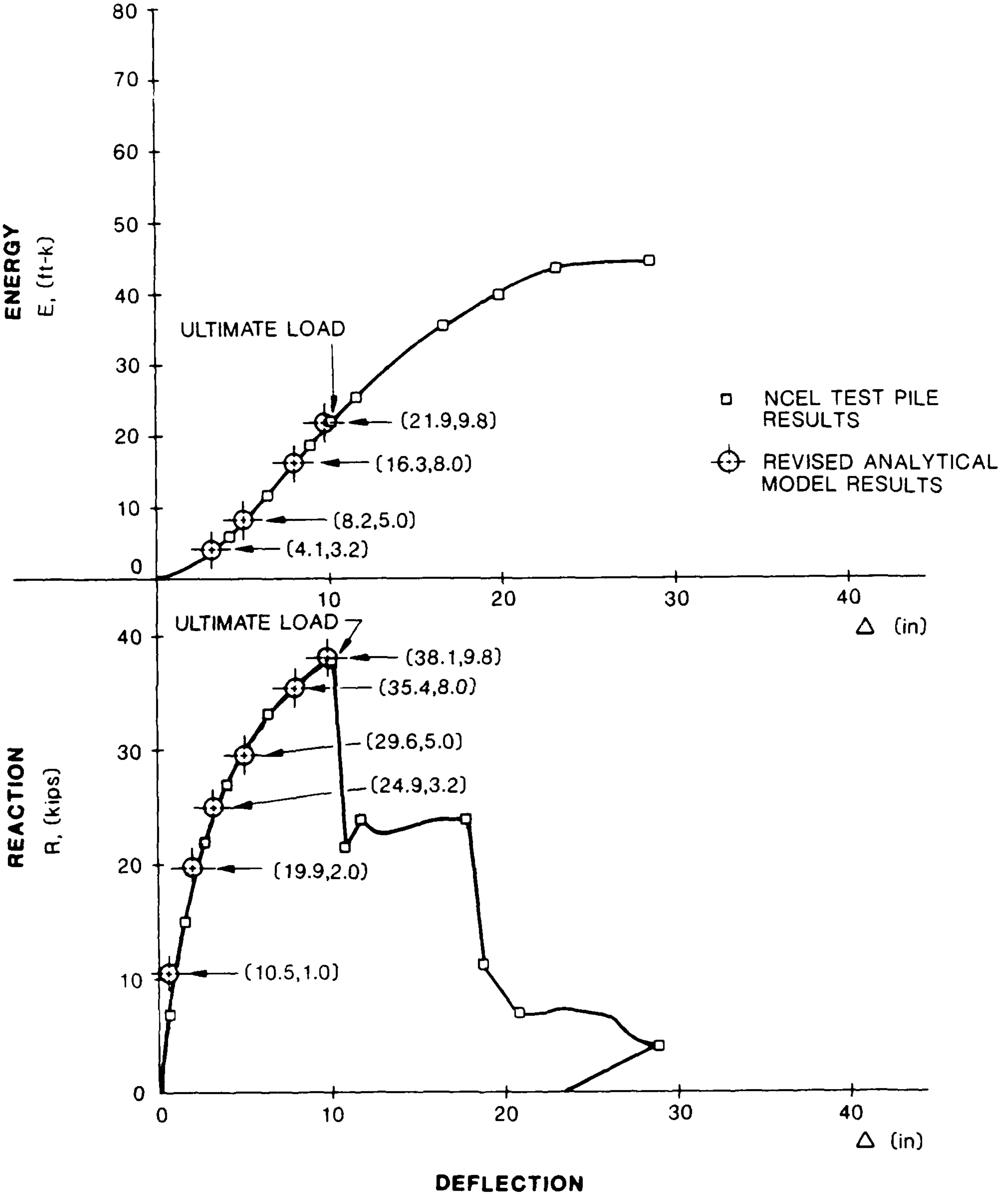


Figure 3.6. Comparison of NCEL Test Results with Test Pile MK-7

3.3 SENSITIVITY STUDY ON ANALYTICAL MODEL PARAMETERS

The discussions above have centered on adjusting the computer model in order to predict the response from NCEL tests. This section will discuss the values recommended for use with the analytical model for design purposes. Although the theory remains the same as that used to match the test results, more conservative parameters are chosen for design.

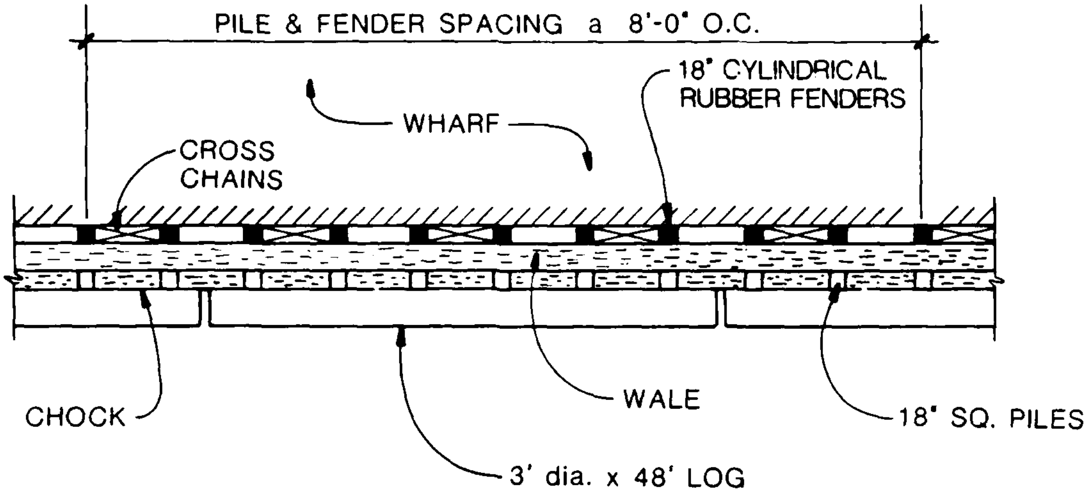
The energy capacity of a pile is dependent on more than just the strength of the cross section. In order to decide which design values and properties to use in the computer program, a sensitivity study was undertaken. This study attempted to show which parameters were important in the design and which ones were not, and also give the designer a better understanding of the effects of tolerances or construction errors. For all the cases, normal weight concrete was assumed.

In this parameter investigation, the pile cross section chosen to represent the baseline case was pile MK-5 from the preliminary tests. Its cross section is shown in Figure 3.7. Table 3.1 lists the reaction, deflection, energy, and change in energy between the baseline case and the particular parameter being investigated. The actual values used in the baseline case are indicated in parentheses in the table. Computer runs were performed keeping everything the same as the baseline case except for changing one parameter at a time. Discussion of the parameters and sensitivity study results is presented next.

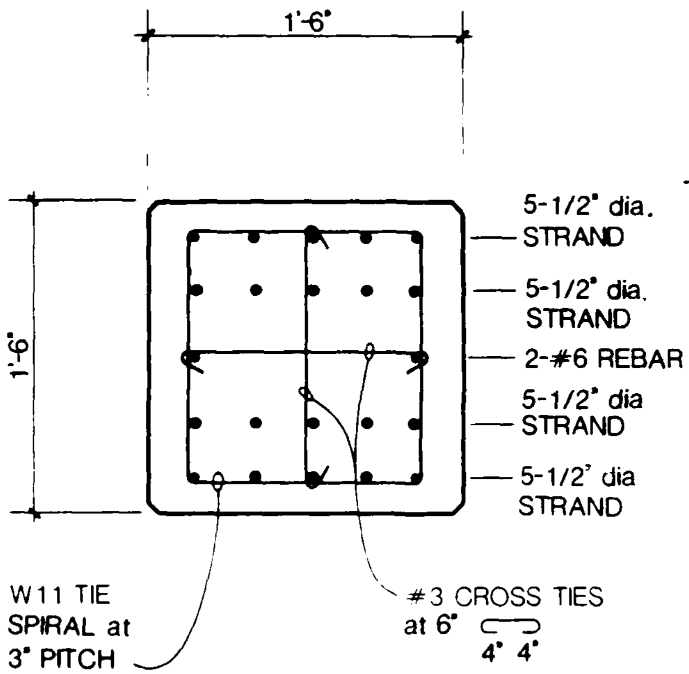
3.3.1 Design Assumptions

a. Concrete Tension

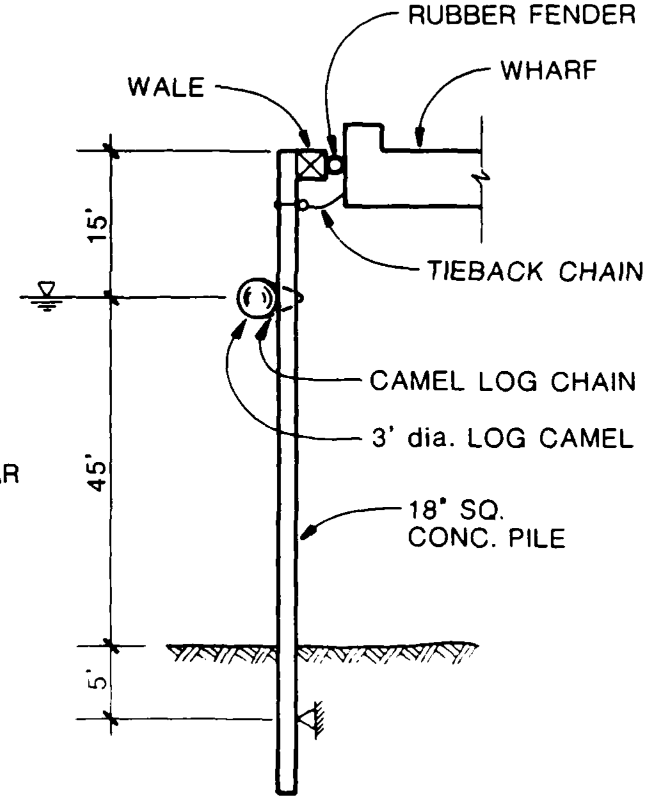
The contribution of concrete tension to the pile capacity is most significant in the lower moment region of the pile.



PLAN - PILES 8'-0" O.C.



CROSS SECTION



SECTION

Figure 3.7. Baseline System and Pile Details

TABLE 3.1
SENSITIVITY STUDY RESULTS

Parameters		R_u (kips)	Δ_u (in.)	E_u (ft-kips)	ΔE %
Baseline Case		34.9	18.2	33.8	0.0
Design assumptions	Value				
Concrete Tension	None	34.9	18.4	33.5	-0.9
	($10\sqrt{f'_c}$)	34.9	18.2	33.8	0.0
ε_u (max) (in./in.)	(0.0003)	34.9	18.2	33.8	0.0
	0.0031	35.3	18.7	35.1	+3.8
	0.0033	36.1	19.5	37.4	+10.7
Pile Length (ft)	60	35.8	16.4	31.2	-7.7
	(65)	34.9	18.2	33.8	0.0
	70	34.2	20.1	36.4	+7.7
Load Location (ft)	10	47.6	13.4	33.8	0.0
	(15)	34.9	18.2	33.8	0.0
	20	29.1	21.9	33.8	0.0
Material properties					
f'_c (ksi)	6	31.0	16.3	27.1	-19.8
	7	33.1	17.3	30.6	-9.5
	(8)	34.9	18.2	33.8	0.0
	9	36.4	19.1	36.8	+8.9
	10	37.6	19.8	39.3	+16.3
f_y (ksi)	(60)	34.9	18.2	33.8	0.0
	70	35.0	18.3	34.0	+0.6
	80	35.2	18.4	34.2	+1.2
E_s (ksi)	27,000	34.7	18.5	34.1	+0.9
	(28,000)	34.9	18.2	33.8	0.0

TABLE 3.1
SENSITIVITY STUDY RESULTS (continued)

Parameters		R_u (kips)	Δ_u (in.)	E_u (ft-kips)	ΔE %
f_{pu} (ksi)	(270)	34.9	18.2	33.8	0.0
	280	35.3	18.5	34.5	+2.1
	290	35.6	18.7	35.2	+4.1
Arbitrary tolerances					
f_{se} (ksi)	50	34.7	19.4	34.9	+3.3
	(60)	34.9	18.2	33.8	0.0
	70	35.1	17.2	32.7	-3.3
Pile Size (in. x in.)	17-1/2 x 17-1/2	34.4	18.0	33.0	-2.4
	(18 x 18)	34.9	18.2	33.8	0.0
	18-1/2 x 18-1/2	35.3	18.5	34.7	+2.7
Rebar Shift (in.)	-1/2	34.7	18.4	33.8	0.0
	(0.0)	34.9	18.2	33.8	0.0
	+1/2	35.1	18.2	33.8	0.0
Strand Shift (in.)	-1/2	33.2	18.8	33.1	-2.1
	(0.0)	34.9	18.2	33.8	0.0
	+1/2	36.6	17.7	34.6	+2.4

Notes

1. Values in parentheses are parameters used in the baseline case.
2. ΔE is percent change from baseline.

Before the concrete cracks, the concrete tension increases the stiffness of the section. As the crack propagates up the section, the tension block also moves closer to the neutral axis. At high moments, the tension block and its moment arm are small, and therefore have little effect on the ultimate capacity of the pile. Its contribution to the pile energy at ultimate is only 0.9%; thus it was considered to be insignificant and will not be used in the program.

b. Maximum Concrete Strain (ε_{cu})

The preliminary test results for test pile MK-5 reported an ultimate concrete strain of 0.0032 in./in. [2.2]. The paper by Pastor [3.3] indicates that high-strength concrete has a higher ultimate strain than typically assumed. The current state of the art is to use a maximum strain of 0.003 in./in. Because this lower strain is conservative and results in a lower ultimate energy, the program uses 0.003 in./in. If the actual strain should be higher, the pile would be capable of absorbing more energy.

c. Pile Length

Figure 3.7 presents a sketch of the fender system used for this study. The depth of the pile point of support below mudline is dependent on the soil conditions and is difficult to evaluate without specific soils information. A variation in the embedment depth of ±5 ft produces a ±7.7% difference in the ultimate energy. Since the ability of the pile to absorb energy increases with an increase in depth to point of support (i.e., longer piles), the designer should be conservative (i.e., shorter pile lengths) when he calculates the point of support of the pile. For the purposes of this study, the span length was limited to 65 ft.

d. Load Location

The location of the applied load does not affect the energy-absorption capacity of the fender pile. The amount of energy that the pile can absorb is essentially limited by the ultimate moment capacity of the section. Generally, the location of the load does not affect the moment capacity of the pile (see Section 4.6). As the pile is impacted closer to the upper support, the load that produces this ultimate moment increases but, because the stiffness is greater, the corresponding deflection decreases. The opposite is true when the pile is impacted farther from the upper support. This inverse relationship results in a constant energy-absorption capacity along the pile. However, the location of the load does affect the stiffness (R/E) and the reaction that the fender pile transmits to the pier. The higher up the load is applied on the pile, the higher the reaction will be on the pier. This, in turn, affects the shear in the pile and the effectiveness of the prestressing strand. These issues will be addressed further in Section 4 of this report.

3.3.2 Material Properties

a. Concrete Strength (f'_c)

Concrete strength is the largest single parameter affecting the energy capacity of the pile. The total pile energy absorption capacity increases approximately 9% when the concrete strength increases from 8000 to 9000 psi. As seen in the preliminary test results, the minimum specified concrete strength was 8000 psi. When the core samples were taken from pile MK-5, they tested out at nearly 10,000 psi. This translates into an increase in energy capacity of over 16%. The opposite would be true if the actual concrete strength were less than that required by specification. Because of the large impact of

concrete strength on pile energy capacity, extra attention should be directed to the mix design and quality control to make sure that the minimum specified concrete strength is attained. The question of whether or not it is unconservative to have a pile with higher than design strength concrete will be addressed in Section 4.5. In all cases, normal weight concrete was assumed.

b. Reinforcing Bar Yield Strength (f_y)

With mild steel reinforcing bars placed near the center of the pile to support the cross ties, the effect of the rebar yield strength on the energy capacity is very small because the internal moment arm of the rebar force is small. For pile design, a specified yield strength of F_y = 60 ksi was used.

c Modulus of Elasticity of Prestressing Strand (E_s)

The modulus of elasticity of the strand has a negligible effect on the pile energy. Per the PCI Design Handbook [3.4], the value of E_s = 28,000 ksi was used in the program.

d Strength of Prestressing Strand (f_{pu})

A higher tensile strength strand does not have a big effect on the energy capacity of the pile for overreinforced sections. As the concrete reaches its ultimate strain, the prestressing strand is just reaching its yield strain. The strand does not have a chance to get much above 240 ksi before the concrete fails. If the pile was underreinforced, the strand would have yielded by the time the concrete reached its compressive limit and more rotation would occur in the section; hence, more energy would be absorbed. For the design program, the minimum specified value of f_{pu} = 270 ksi was used.

3.3.3 Tolerances

a. Effective Stress in Prestressing Strands (f_{se})

This was included in the study to look at the effect of either the contractor stressing to the wrong initial stress or the long-term losses in the strand being different than originally calculated. The analysis indicated only a ±3.3% change in energy capacity for a ∓10-ksi change in effective prestress.

b. Pile Size

In order to remove the pile from the form, the contractor may have to provide a slight draft in the form, which would change the dimensions of the pile used in the design. A ±1/2-in. change in nominal pile dimensions results in a ±2.7% change in energy capacity.

c. Shift in Reinforcing Bar Location

A shift of the mild steel reinforcing bar relative to the center of the pile could be due to either tying it in the wrong location or the entire rebar cage shifting during the concreting operations. With only two No. 6 bars at middepth of the section, the effect of a ±1/2-in. (- = movement toward the compression/top face, + = movement toward the tension/bottom face of the cross section) movement of the rebar is negligible.

d. Shift in Strand Location

Because the strand is pretensioned to a much lower prestress level than in typical prestress members (60- versus 189 ksi), there could be a sag at midspan unless the strand is adequately supported and tied off to the confinement reinforcement. A ±1/2-in. (- = movement toward the compression/top face,

+ = movement toward the tension/bottom face of the cross section) shift in the strand location results in a ±2.4% change in energy.

3.4 SUMMARY OF ANALYTICAL COMPUTER MODEL

3.4.1 Summary of Parameters Embedded in Program

The following is a brief description of the parameters or assumptions embedded in the computer program FENDER. The user must assume responsibility if any of these are changed without thoroughly checking the results.

a. Normal weight concrete is used.

b. The maximum allowable strain in the concrete is 0.003 in./in.

c. The concrete is assumed to have no tension capacity.

d. The concrete stress block is trapezoidal in shape at high concrete strains.

e. The Morales equation is used for the modulus of elasticity of the concrete.

f. The minimum specified properties for the mild reinforcing steel are

- o Yield strength, f_y = 60 ksi
- o Modulus of elasticity, E_s = 29,000 ksi

g. The minimum specified properties of the prestressing strand are

- o Specified tensile strength of strand, f_{pu} = 270 ksi

- o Yield stress for stress-relieved strand, $f_{py} = 0.85 \times f_{pu}$, and low relaxation strand, $f_{py} = 0.90 \times f_{pu}$
- o Modulus of elasticity, $E_s = 28,000$ ksi

3.4.2 Input Variables Required

The following is a list of variables that are required in the input statement:

a. The rectangular dimensions of the concrete cross section

b. The total design length from the centerline of the upper support of the pile to the assumed point of support below mudline

c. The distance from the upper support to where the pile is assumed loaded

d. The specified concrete strength, f'_c

e. The area of the mild reinforcing steel and its distance from the compression face of the pile

f. The type of strand being used, either stress relieved or low relaxation

g. The effective prestress level in the strands, f_{se}

h. The area of the prestressing steel and its distance from the compression face of the pile

3.4.3 Sample Computer Input/Output

Table 3.2 shows a typical input file for a pile with 16 strands. This particular pile will be discussed further in Section 4 as being the most efficient pile in this study for the target energy level of 30 ft-kips. Figure 3.8 shows the output from the FENDER computer run for this same pile.

TABLE 3.2
PROGRAM "FENDER" INPUT

```
NCEL Prestressed Fender Pile Study
18-in.-square rectangular strand pattern, 16 strands @ 59.6 ksi 6/2
65, 15
18, 18, 8
1
0.88, 9.0
LR
16, 59.6
4
0.918, 2.75
0.306, 4.75
0.306, 13.25
0.918, 15.25
```

Note: For a discussion of the input file, refer to Appendix C.

```
*          PROJECT:        NCEL PRESTRESSED FENDER PILE STUDY
*          DATE:           11/20/85
*          DESIGNER:       CWS
*          PROGRAM:        'FENDER' DEVELOPED BY ABAM ENGR. INC.
*
*          DATA FILE:      FEN16.DAT
*          PILE NUMBER:    18" SQUARE RECTANGULAR STRAND PATTERN
*          REVISION #:     0
*          REMARKS:        16 STRANDS @ 59.6 ksi 6/2
*
*          PILE DIMENSIONS:
*                          LENGTH          = 65 ft
*                          LOAD FROM TOP   = 15 ft
*                          DEPTH           = 18 in
*                          WIDTH           = 18 in
*          CONCRETE DATA:
*                          STRENGTH f'c    = 8 ksi
*                          MODULUS E       = 4577 ksi
*                          PRESTRESS fpc   = 450 psi
*          REBAR DATA:
*                          YIELD fy        = 60 ksi
*                          MODULUS E       = 29000 ksi
*                          AREA = .88 sq.in             DEPTH = 9 in
*          STRAND DATA:
*                          STRAND TYPE     = LOW RELAXATION
*                          # STRANDS       = 16
*                          STRESS fpu      = 270 ksi
*                          STRESS fse      = 59.6 ksi
                           MODULUS E       = 28000 ksi
                           AREA = .918 sq.in            DEPTH = 2.75 in
*                          AREA = .306 sq.in            DEPTH = 4.75 in
*                          AREA = .306 sq.in            DEPTH = 13.25 in
*                          AREA = .918 sq.in            DEPTH = 15.25 in
```

MOMENT Mn (k-ft)	CURVATURE (rad/in)	CONC STRAIN (in/in)	UPPER STRAND STRESS (ksi)	LOWER STRAND STRESS (ksi)	NA DEPTH (in)
38.0	1.075E-05	0.00019	-57.8	-61.5	18.00
66.5	2.541E-05	0.00030	-55.9	-64.8	11.81
97.9	6.435E-05	0.00050	-53.3	-75.8	7.77
125.8	1.115E-04	0.00070	-51.3	-90.3	6.28
153.8	1.624E-04	0.00090	-49.7	-106.5	5.54
182.4	2.153E-04	0.00110	-48.1	-123.5	5.11
211.2	2.694E-04	0.00130	-46.7	-141.0	4.83
240.3	3.241E-04	0.00150	-45.3	-158.8	4.63
266.6	3.763E-04	0.00170	-43.7	-175.5	4.52
289.1	4.247E-04	0.00190	-41.9	-190.5	4.47
308.8	4.715E-04	0.00210	-39.9	-204.9	4.45
325.8	5.192E-04	0.00230	-37.9	-218.6	4.43
338.7	5.683E-04	0.00250	-36.1	-228.2	4.40
348.8	6.186E-04	0.00270	-34.4	-235.0	4.36
356.4	6.705E-04	0.00290	-32.8	-240.1	4.32
359.4	6.971E-04	0.00300	-32.0	-242.1	4.30

Figure 3.8. Sample Output

* DATA FILE: FEN16.DAT
* PILE NUMBER: 18" SQUARE RECTANGULAR STRAND PATTERN
* REVISION #: 0
REMARKS: 16 STRANDS @ 59.6 Ksi 6/2

MOMENT Mn (k-ft)	CONC STRAIN (in/in)	ENERGY (k-ft)	LOAD (kips)	DEFLECTION (in)
38.0	0.00019	0.1	3.3	0.4
66.5	0.00030	0.2	5.8	0.8
97.9	0.00050	0.8	8.5	1.8
125.8	0.00070	1.8	10.9	3.0
153.8	0.00090	3.3	13.3	4.6
182.4	0.00110	5.4	15.8	6.2
211.2	0.00130	7.9	18.3	8.0
240.3	0.00150	10.9	20.8	9.9
266.6	0.00170	14.1	23.1	11.6
289.1	0.00190	17.2	25.1	13.1
308.8	0.00210	20.2	26.8	14.5
325.8	0.00230	23.0	28.2	15.8
338.7	0.00250	25.5	29.4	16.8
348.8	0.00270	27.7	30.2	17.7
356.4	0.00290	29.5	30.9	18.4
359.4	0.00300	30.3	31.2	18.7

Figure 3.8. Sample Output (continued)

SECTION 4
PILE OPTIMIZATION

4.1 BASIS OF PILE OPTIMIZATION

This study was focused on designing the most efficient prestressed concrete pile for a given energy level. The object in designing an efficient pile is to minimize the number of strands and the prestress level in the strands in order to minimize the pile cost and maximize the available stress range. The ultimate energy requirement for one concrete pile is 30 ft-kips. This represents an upper bound energy that a concrete pile should see in a typical Navy fender system excluding fenders for carrier piers. It also represents the same target energy level used in the Phase I study for consistency in analysis and testing. From the results of the preliminary pile testing, it was determined that piles with a rectangular reinforcing pattern were the most efficient [2.2]. The 18-in.-square pile size and a concrete strength of f'_c = 8000 psi are also carried over from the Phase I report. Other combinations of pile size and concrete strength were briefly reviewed. These will be discussed in Section 4.5.

Figure 4.1 shows the load-deflection curves for three prestressed concrete piles that are exactly the same except for the initial prestress level in the strands. The pile with 0.0-ksi prestress represents the theoretical upper limit for energy-absorption capacity, but it is not practical because of handling and driving stress requirements (see Section 4.2). Because the available elastic stress range in the strand is about 240 ksi, this pile is able to absorb 35.1 ft-kips of energy. The lower bound is represented by the pile with a 150-ksi effective prestress. This is a typical

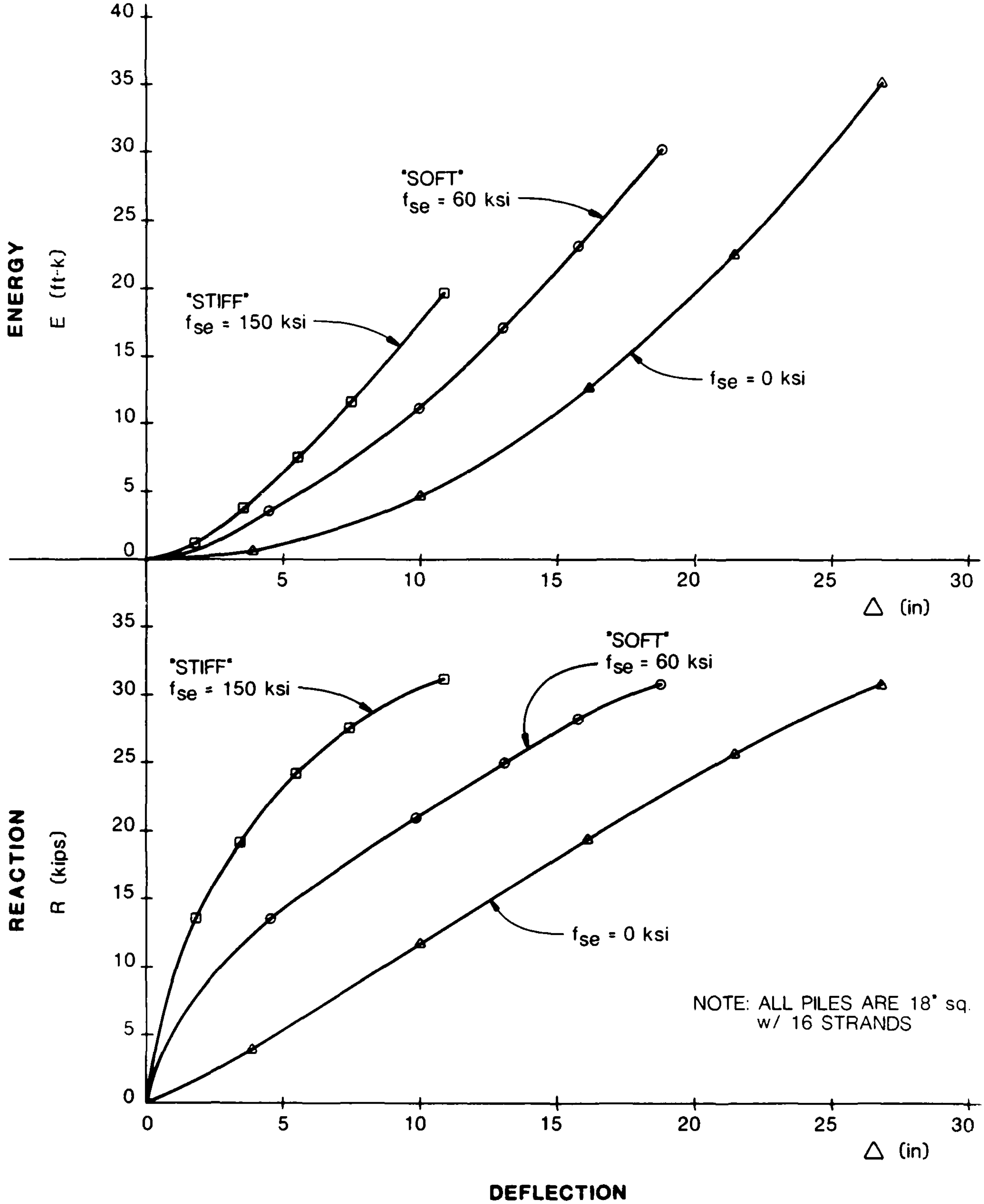


Figure 4.1. Comparisons of Initial Prestress

prestress level, after all losses, for most pier support piles but only offers a 240 - 150 = 90-ksi stress range. This results in a very stiff pile which only absorbs 19.6 ft-kips of energy. The third pile has 60 ksi of prestress in the strand and represents the level on which we have concentrated in this phase. The allowable stress range is still large, 240 - 60 = 180 ksi, but there is sufficient prestress to control driving and handling stresses. The energy capacity is also relatively high at 30.3 ft-kips.

4.2 MINIMUM PRESTRESS LEVEL (f_{pc})

Two critical items that limit the minimum prestress level in the pile are handling and driving stresses. Initially, a minimum prestress level of 500 psi ±50 psi was chosen for an 18-in.-square pile. An 80-ft total pile length was investigated as being representative of a maximum length of fender pile that may be constructed. A two-point pick was considered for the pile, with an impact factor of 1.5. The resulting maximum moment in the pile was less than the cracking moment. Even if the resulting moment were slightly greater than the cracking moment and the pile cracked, the prestress force would close the cracks once the pile was in an upright (vertical) position or unloaded. The very purpose of the fender pile is to deflect and absorb energy and it is expected that the pile will crack at low energy levels but that the cracks will close after removal of the load. The driving forces in the fender piles are expected to be minimal in most instances. In addition, proper softwood cushions must be used under the driving head to prevent tensile wave cracking. A hypothetical case was considered where the pile is driven into soft soil with a 15-ft-kip rated hammer. Based on our analysis for a pile with a 450-psi prestress level, the driving presented no problems to the pile. Based on the above, a minimum concrete prestress level of 450 psi appears acceptable for an 18-in.-square pile.

4.3 PILE SIZE INVESTIGATION

Before a full analysis was performed on 18-in.-square piles for the 30-ft-kip energy level, smaller size piles with 8000-psi concrete were investigated. The 14- and 16-in.-square piles both had an initial concrete stress of 500 psi. The 14-in. pile had an energy level of 18.2 ft-kips while the 16-in. pile had an energy of 25.5 ft-kips. For the 16-in. pile, the concrete strength was then increased to 9000 psi. Even with 24 strands, the energy level was only 28.2 ft-kips. Both size piles fell short of the target 30 ft-kip energy level and were not investigated further. In future phases of the development, smaller size piles should be investigated for applications having lower energy level requirements.

4.4 RESULTS OF STRAND PATTERN INVESTIGATION

Having established a minimum prestress level in the concrete of 450 psi, using a pile size of 18-in.square and choosing a minimum energy level of 30 ft-kips, the prestressing strand in the pile was optimized. Based on the above constraints, our approach was to reduce the number of strands until a minimum number was found below which the pile could not absorb 30 ft-kips of energy. Five different types of strand "layouts" were investigated and are summarized below:

- 1/2-in.-diameter strands in a rectangular pattern
- 1/2-in.-diameter strands in a circular pattern
- 0.6-in.-diameter strands in a rectangular pattern
- 1/2-in.-diameter strands in a rectangular pattern with unstressed strands added
- Miscellaneous strand patterns

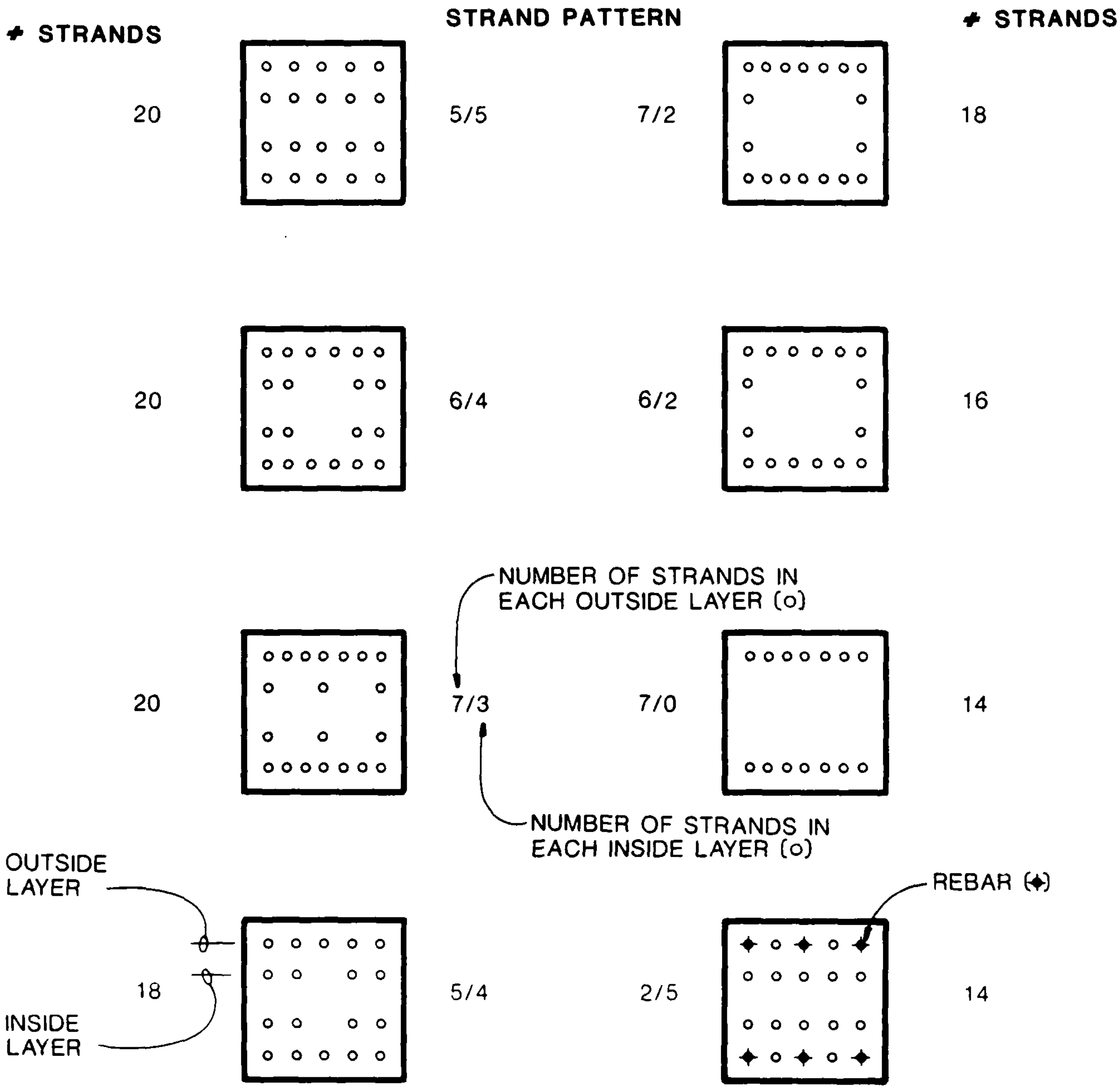
A sketch of each strand "pattern" reviewed is shown in Figures 4.2, 4.3, and 4.4. Computer runs were then performed and the results are summarized in Table 4.1. Following is a discussion of the different strand layouts.

4.4.1 1/2-In.-Diameter Rectangular Pattern

Figure 4.2 shows the seven different strand patterns investigated using 1/2 -in. -diameter strands in a rectangular pattern. The starting point was a 20-strand pattern similar to the baseline pile MK-5 in the preliminary tests. This configuration, with a 7/3 pattern, has an energy capacity of 32.9 ft-kips.

The optimum strand configuration without mild steel reinforcing has 16 strands in a 6/2 pattern at the top and bottom face of the pile. A 450-psi concrete prestress level was used and resulted in an energy capacity of 30.3 ft-kips. This strand pattern was chosen to allow installation of a through bolt between the strands with a maximum amount of the cover to adjacent strands for corrosion protection. This strand configuration was also the optimum for all the types of strand patterns investigated. See Figure 2.1 for details.

The pile configuration that required the minimum number of strands (14) in order to meet the 30-ft-kip energy level also had three No. 8 mild steel reinforcing bars on both the top and bottom faces. There are possible disadvantages to placing mild steel reinforcing near the top and bottom faces of the pile. First, the mild steel rebar will reach its yield point at a low energy level of about 9.4 ft-kips. As the pile continues to absorb energy, the mild steel rebar on the tension face of the pile will begin to "neck down" due to Poisson's effect. The rebar on the compression face will also yield, but in compression. The effect of cyclic yielding of the mild steel reinforcing needs further investigation should this strand pattern be used. Another potential problem is the serviceability of the pile. Once the mild steel rebar has yielded, it may hinder the cracks from closing up once the load is removed.



STRAND SIZE: 1/2" dia.
CONFIGURATION: RECTANGULAR

Figure 4.2. Pile Strand Patterns

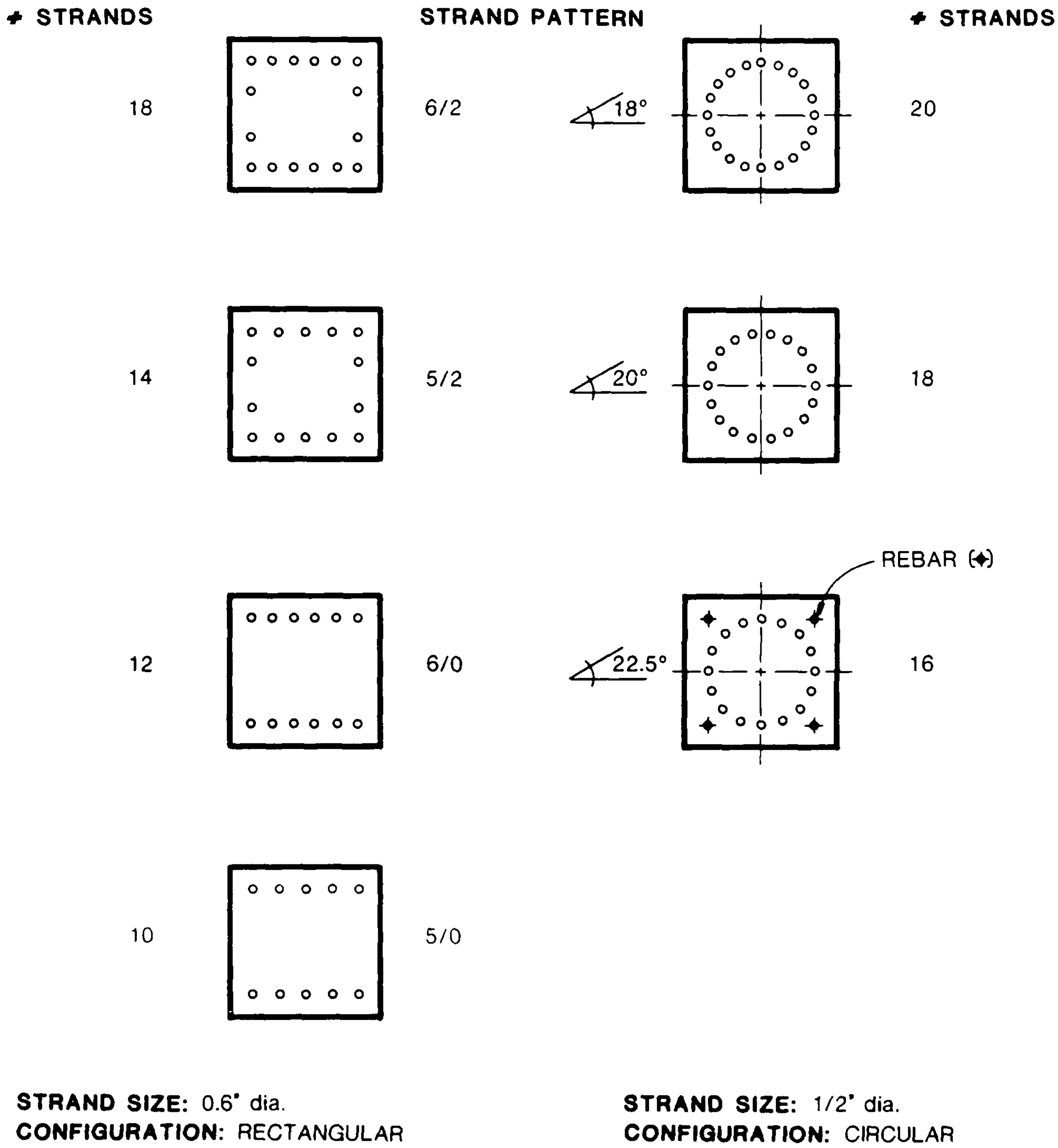


Figure 4.3. Pile Strand Patterns

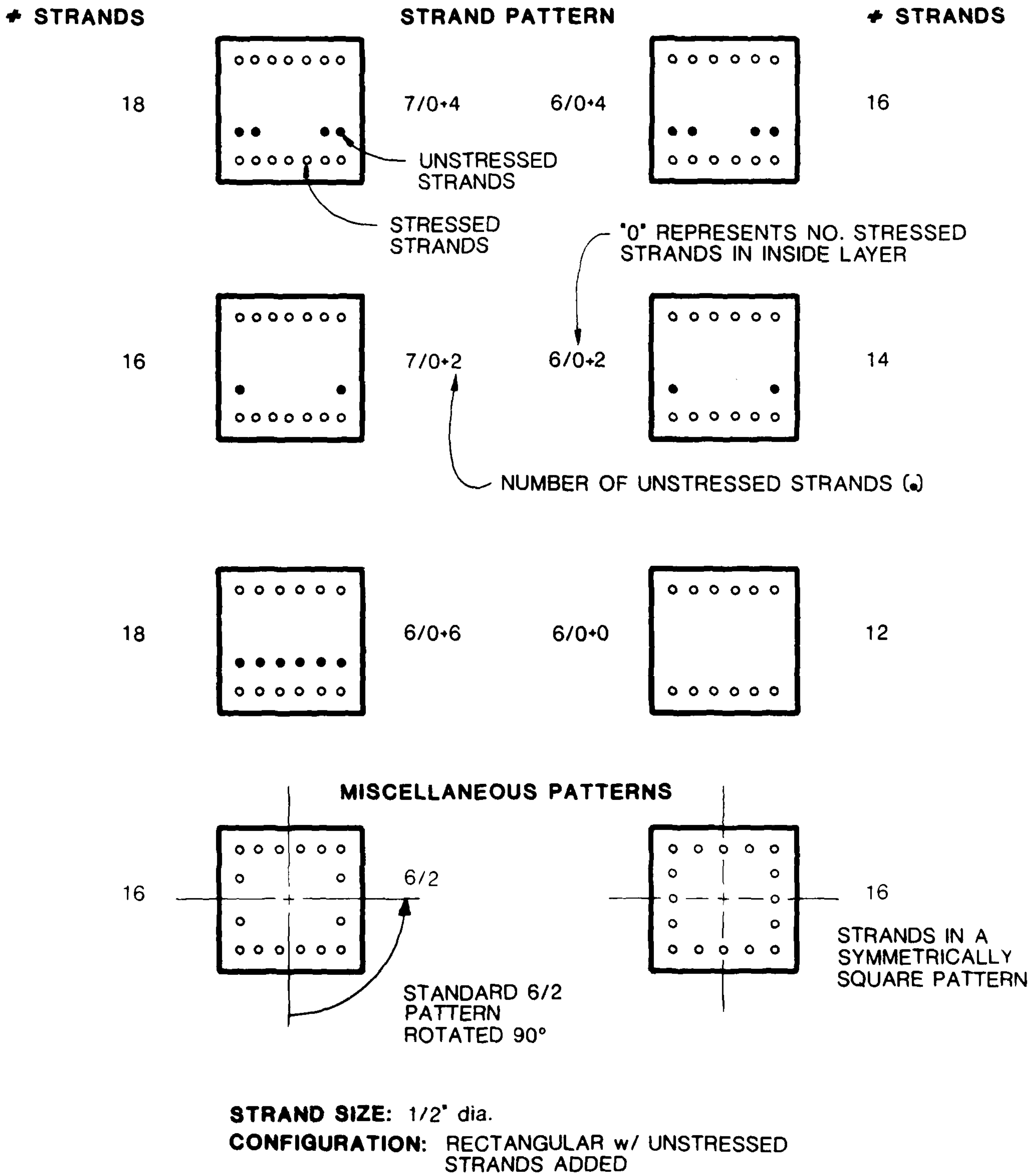


Figure 4.4. Pile Strand Patterns

TABLE 4.1
STRAND PATTERN INVESTIGATION RESULTS

No. of Strands	Strand Pattern	f_{se} (ksi)	f_c (psi)	R_u (kips)	E_u (ft-kips)
1/2-in. strand, rectangular pattern					
20	5/5	60.0	566	33.6	30.8
20	5/5	53.0	500	33.5	31.6
20	6/4	53.0	500	34.2	32.3
20	7/3	53.0	500	35.0	32.9
18	5/4	60.0	510	32.3	30.6
18	5/4	53.0	450	32.2	31.4
18	7/2	60.0	500	33.6	31.7
18	7/2	53.0	450	33.5	32.5
16	6/2	66.2	500	31.2	29.6
16	6/2	59.6	450	31.2	30.3
14	7/0	75.7	500	29.5	27.3
14	7/0	68.1	450	29.5	28.2
14*	2/5	68.1	450	35.5	30.0
1/2-in. strand, circular pattern					
20	Circular	53.0	500	29.4	29.5
20	Circular	47.7	450	29.3	30.0
18	Circular	58.9	500	28.0	28.5
18	Circular	58.9	450	27.8	29.1
16**	Circular	66.2	500	30.6	28.9
16**	Circular	59.6	450	30.4	29.7
0.6-in. strand, rectangular pattern					
16	6/2	47.0	500	37.0	34.3
16	6/2	42.4	450	36.9	34.9
14	5/2	53.8	500	34.3	32.4
14	5/2	48.4	450	34.3	33.0
12	6/0	62.8	500	33.5	31.5
12	6/0	56.5	450	33.5	32.3
10	5/0	75.3	500	29.6	27.4
10	5/0	67.8	450	29.6	28.3

* Total of six No. 8 bars
** No. 6 bar in each corner of pile

TABLE 4.1
STRAND PATTERN INVESTIGATION RESULTS (continued)

No. of Strands	Strand Pattern	f_{se} (ksi)	f_c (psi)	R_u (kips)	E_u (ft-kips)
1/2-in. strand, rectangular pattern with unstressed strands					
14 + 4	7/0 + 4	68.1	450	35.1	32.7
14 + 2	7/0 + 2	68.1	450	32.8	31.3
12 + 6	6/0 + 6	79.5	450	35.0	31.9
12 + 4	6/0 + 4	79.5	450	32.9	31.0
12 + 2	6/0 + 2	79.5	450	30.2	29.2
12 + 0	6/0 + 0	79.5	450	26.5	24.9
Miscellaneous strand patterns					
Pile rotated 90°					
16	6/2	59.6	450	29.1	29.5
Pile with square strand pattern					
16	Square	59.6	450	27.9	29.2

4.4.2 1/2-In.-Diameter/Circular Pattern

Figure 4.3 shows the circular patterns investigated for the 1/2-in.-diameter strands. A total of 20 strands were required in order to meet the energy requirement of 30 ft-kips. When mild steel reinforcing is added to the corner of the pile, the number of strands required was reduced to 16, with the resulting energy equaling 29.7 ft-kips (just short of 30 ft-kips).

4.4.3 0.6-In.-Diameter/Rectangular Pattern

Rather than using the typical 0.5-in.-diameter prestressing strand, another possibility is to use the larger 0.6-in.-diameter strand. Because of its greater area, fewer number of strands are required to provide the same force as the 0.5-in.-diameter strand. The minimum strand configuration consisted of 12 strands in a 6/0 pattern at the top and bottom of the cross section. The energy capacity with a 500-psi concrete prestress level was 31 ft-kips. The advantage to using 0.6-in.-diameter strand would be that fewer strands would have to be handled and jacked, thereby possibly saving on labor costs. The disadvantages to using 0.6-in.-diameter strands are that it is more difficult to handle, not as readily available as 0.5-in.-diameter strand, and fewer precasters use it. With this in mind, we are not pursuing it any further, but it remains a possible alternative under the right circumstances.

4.4.4 1/2-In.-Diameter Unstressed Strands Added

The third type of strand pattern investigated uses a combination of stressed and unstressed 1/2-in.-diameter strands. These patterns are shown in Figure 4.4. The optimum pile for this condition uses 12 stressed strands equally distributed both top and bottom, plus four unstressed strands on the tension face only to cut down on the total number of strands. The resulting energy capacity was 31.5 ft-kips. In order to meet the requirement for a minimum prestress in

the concrete of 450 psi, the tensioned strands must be initially stressed to 80 ksi versus 60 ksi for the typical case, thereby resulting in a slightly stiffer pile. A possible difficulty in placing unstressed strands on one side only is that the pile must be installed in the correct orientation. On the other hand, if unstressed strands were also added to the compression face, they would act as compression steel to make the pile even stiffer.

4.4.5 Miscellaneous Strand Patterns

Figure 4.4 shows the miscellaneous strand patterns investigated. The most efficient pile was the 16-strand pile with a 6/2 strand pattern, as described in Section 4.4.1. This pile uses the fewest number of strands yet meets the energy requirement for 30 ft-kips. Since the pile is only symmetrical about one axis, a computer run was performed to determine the pile energy capacity if the pile were accidentally rotated 90 degrees during installation. The resulting energy was reduced from 30.3 to 29.2 ft-kips, which would be acceptable in the event that the pile is installed incorrectly.

We also considered the possibility of using the same 16 strands and placing them in a square, symmetrical strand pattern around the section. The resulting energy capacity was 29.2 ft-kips, which is less than but close to the required design energy of 30 ft-kips. Based on actual energy requirements for design at a particular site, and to alleviate the pile orientation requirements, a square pattern of strand may be considered in the design.

4.4.6 No Mild Steel Reinforcing

The two No. 6 mild reinforcing reinforcing bars at middepth of the section are not required for strength or energy capacity of the pile. Their sole purpose is to provide anchorage for the cross ties. In the event that it is determined, by testing, that the cross ties are not required, these reinforcing bars can be omitted.

The resulting pile would have a small decrease in stiffness, which results in a slightly higher energy capacity.

4.5 CONCRETE STRENGTH INVESTIGATION

As first mentioned in Section 3.2, the actual concrete strength in the pile may be considerably different than specified. This raised the question of whether or not this unexpected increase or decrease in strength would have a deterimental effect on the pile.

In the sensitivity study (Section 3.3), concrete strengths of 6,000, 7,000, 8,000, 9,000, and 10,000 psi were investigated for the same pile configuration. Figure 4.5 is a plot of the results of FENDER computer runs showing the reaction versus energy (R/E) curve for each concrete strength. Notice that all the curves are virtually the same except for their individual termination points. The important conclusion to be drawn from this graph is that the R/E ratio for the pile does not vary significantly with concrete strength; however, the ultimate energy capacity and reaction increase with higher concrete strengths. In other words, for a given energy level in the pile, it does not matter if the concrete strength is greater than required as the corresponding reaction will be essentially the same.

In addition to the optimization of the 18-in.-square piles with a concrete strength of 8000 psi, we also investigated the possibility of using the same size pile with a lower strength concrete. The purpose was to reduce construction costs. A pile with 6000-psi concrete could not absorb 30 ft-kips of energy, regardless of the number of strands used. A pile with 7000-psi concrete could absorb the required energy, but it would require 22-1/2-in.-diameter strands. The required increase in strands from 16 to 22 would more than offset the cost saved by utilizing a lower strength concrete. However, if 8000-psi concrete is not available, a solution, albeit more expensive, does exist.

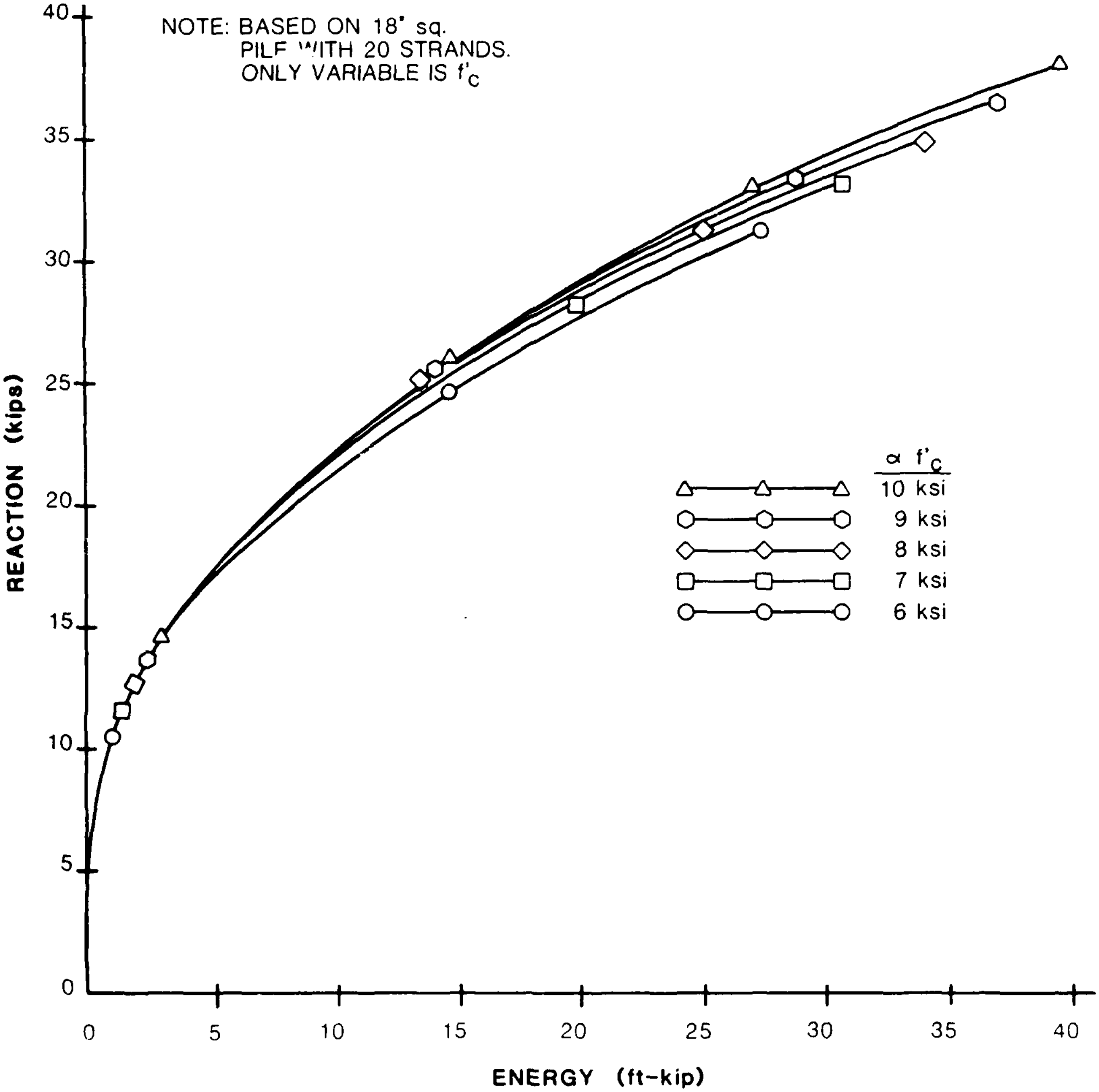


Figure 4.5. Effects of Concrete Strength on Reaction and Energy

4.6 EFFECTS OF LOCATION OF APPLIED LOAD ON PILE

In this analysis, we assumed that the rubber fender at the top of the support did not affect the energy requirement of the pile. In actuality, the rubber fender will take a larger percentage of the total energy as the load is applied closer to the upper support. For comparison purposes, an energy capacity requirement of 30 ft-kips in the pile will be considered, regardless of where it is loaded.

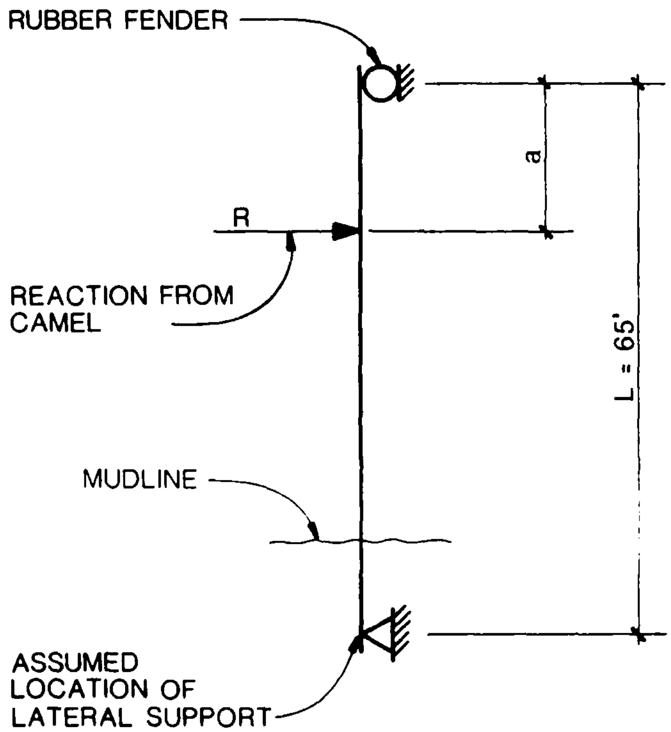
The location of the load relative to the support affects the design of the pile in two different ways: pile shear and pile moment capacities. The shear force that the pile must resist, as was mentioned in Section 3.3.1.d and shown in Table 4.2, varies with the location of the load. The location does not affect the amount of energy that the pile can absorb but it does affect the reaction the pile must resist for a given energy level. The closer the pile is loaded to the upper support, the larger the corresponding shear force in the pile becomes. The critical shear force is therefore dependent on how close to the support the pile is loaded.

The moment capacity of the pile, as shown in Table 4.2, is not affected by where the pile is loaded. The critical moment is at the load application point. But, in order to provide the necessary moment capacity, the strands must be fully developed at that section. Since the effective prestress is low (approximately 60 ksi), the development length required by the ACI code [3.1] is approximately 8 ft 6 in. This requirement must be addressed if the pile is loaded within 10 ft of the support.

TABLE 4.2
LOCATION OF APPLIED LOAD ON PILE

a (ft)	M_n (kip-ft)	E_u (ft-kips)	Δ_u (in.)	R_u (kips)
5.0	359	30.3	7.5	77.9
7.5	359	30.3	10.8	54.2
10.0	359	30.3	13.7	42.5
12.5	359	30.3	16.4	35.6
15.0	359	30.3	18.7	31.2
17.5	359	30.3	20.8	28.1
20.0	359	30.3	22.5	26.0

Note: Based on 18-in.-square piles with 16 strands and design value material properties



4.7 LOWER STRENGTH STRAND (f_{pu} = 250 ksi)

The most common strand on the market has a tensile strength of f_{pu} = 270 ksi. The computer analysis program FENDER is set up to run only strand with f_{pu} = 270 ksi, but it could be modified in the future to run strand with f_{pu} = 250 ksi. The effect of this lower tensile strength would be a reduced energy capacity. See Appendix C for further details.

SECTION 5
COMPARATIVE COST ANALYSIS

Our work in this section compares concrete, steel, and timber fender piles. The approach taken was to investigate a fender pile from the baseline fender system (see Figure 3.7) for a specific energy level for all three materials. The concrete piles were designed using an ultimate strength design approach being developed in this research program, while current practice requires steel and timber piles to be designed by working stress methods. Because no load factor has yet been defined for ultimate strength designs of prestressed concrete fender piles, reasonable assumptions must be made to establish a reasonable working energy level in the concrete pile. The ultimate energy level of 30 ft-kips per pile, is the same as used in the rest of this report. Figure 5.1 shows relatively equivalent energy-absorbing piles.

5.1 CONCRETE PILES

The concrete pile was designed by ultimate strength design methods. In order to compare the concrete with the steel and timber, a working energy was calculated that was lower than 30 ft-kips. Ultimate strength design of concrete requires use of a "phi" factor and a load factor. Phi (ϕ) is the strength reduction factor that takes into account the possible reduction in strength due to both workmanship and material properties. As seen in the results of the NCEL tests, the actual material properties are often much higher than the minimum specified properties. Also, the workmanship in a precast plant is much better than cast-in-place concrete work. Per the 1983 AASHTO Specifications (American Association of State Highway and Transportation Officials), Section 9.14 [5.1], for factory-produced precast prestressed concrete, the recommended ϕ-factor is 1.0.

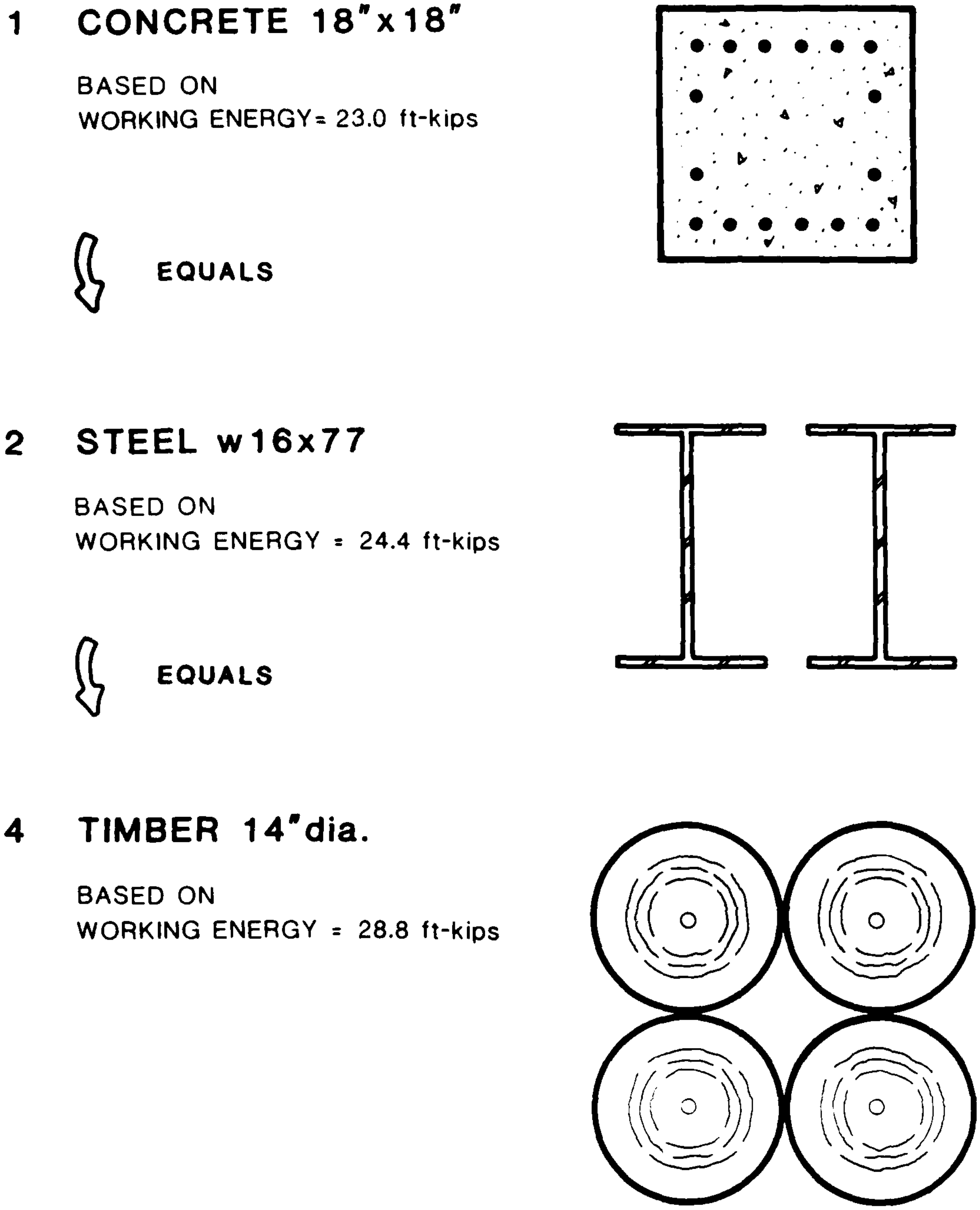


Figure 5.1. Equivalent Energy-Absorbing Piles

Based on these considerations, a ϕ-factor of 1.0 was used in this study.

The choice of a load factor is much harder to make. Figure 5.2 shows a comparison of the R-Δ behavior for various pile materials. Ultimate energy capacity was not investigated in great detail for steel and timber piles because of anticipated permanent set in the steel and lack of reference data on timber pile behavior beyond the proportional limit. The energy calculated for a ship in the Navy's DM 25.1 manual is neither an everyday working energy nor a once-in-a-lifetime event. For the purposes of this report, a load factor of 1.3 was chosen, based on our judgment that the ship impact loading is of short duration, similar to a wind or earthquake loading. The load factor used in the ACI code for wind and earthquakes varies between 1.28 (0.75 x 1.75) and 1.40 (0.75 x 1.75 x 1.1). A higher load factor does not appear warranted for application in the design of a fender pile. This results in a working energy level of 30.0 x 1.0/1.3 = 23.0 ft-kips for the prestressed concrete piles.

5.2 STEEL PILES

Steel piles, for comparative purposes, were selected using working stress design methods to achieve 23 ft-kips or greater of energy. ASTM A 572 steel, with a yield strength of 50 ksi, was used. For an adequately braced compact section, with a short duration load (berthing load similar to wind or seismic), the allowable design stress is $(0.66 \times f_y) \times 1.33 = 0.89 \times f_y$. Three different types of steel shapes were investigated: wide flange, pipe, and structural tubes. The pipe and tube sections were both more costly and stiffer than the wide flange section; hence, they have a lower energy for a given reaction. Two W16x77 WF sections provide an energy capacity of 24.4 ft-kips, which is approximately equal to one 18-in.-square prestressed concrete fender pile. This assumes that sufficient walers are provided to adequately brace the WF sections.

CONCRETE

ULTIMATE DESIGN

PHI ($\emptyset$) = 1.0

LOAD FACTOR = 1.3

$E_u \approx 0.62\ R_d \Delta_d$

WORKING ENERGY

$E_w = E_u \times 1.0/1.3$

STEEL

ALLOWABLE DESIGN STRESS

$f_b = 0.89\ f_y$

WHERE f_y = YIELD STRESS

f_b = BENDING STRESS

WORKING ENERGY

$E_w = 0.5\ R_d \Delta_d$

TIMBER

ALLOWABLE DESIGN STRESS

$f_b = 0.7\ f_r$

WHERE f_b = BENDING STRESS

f_r = MODULUS OF RUPTURE

WORKING ENERGY

$E_w = 0.5\ R_d \Delta_d$

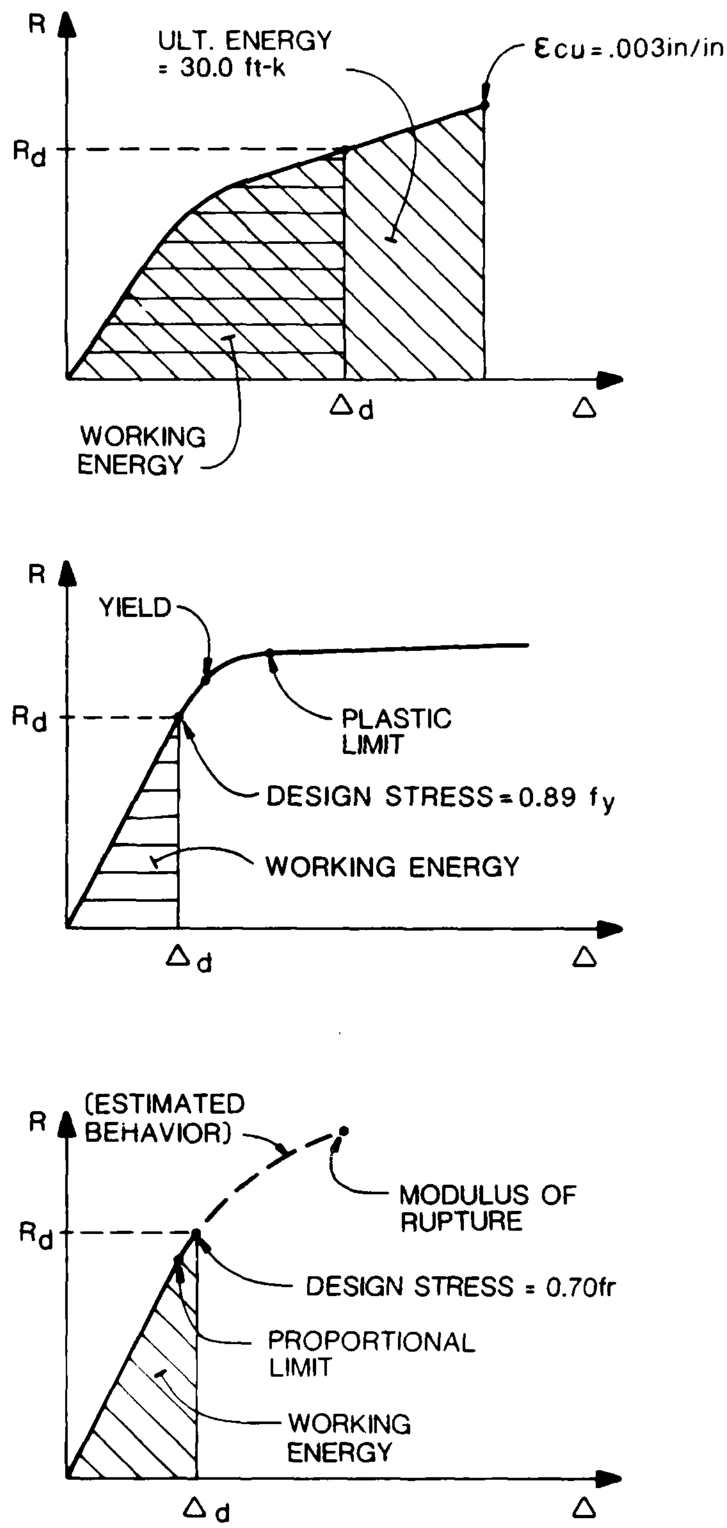


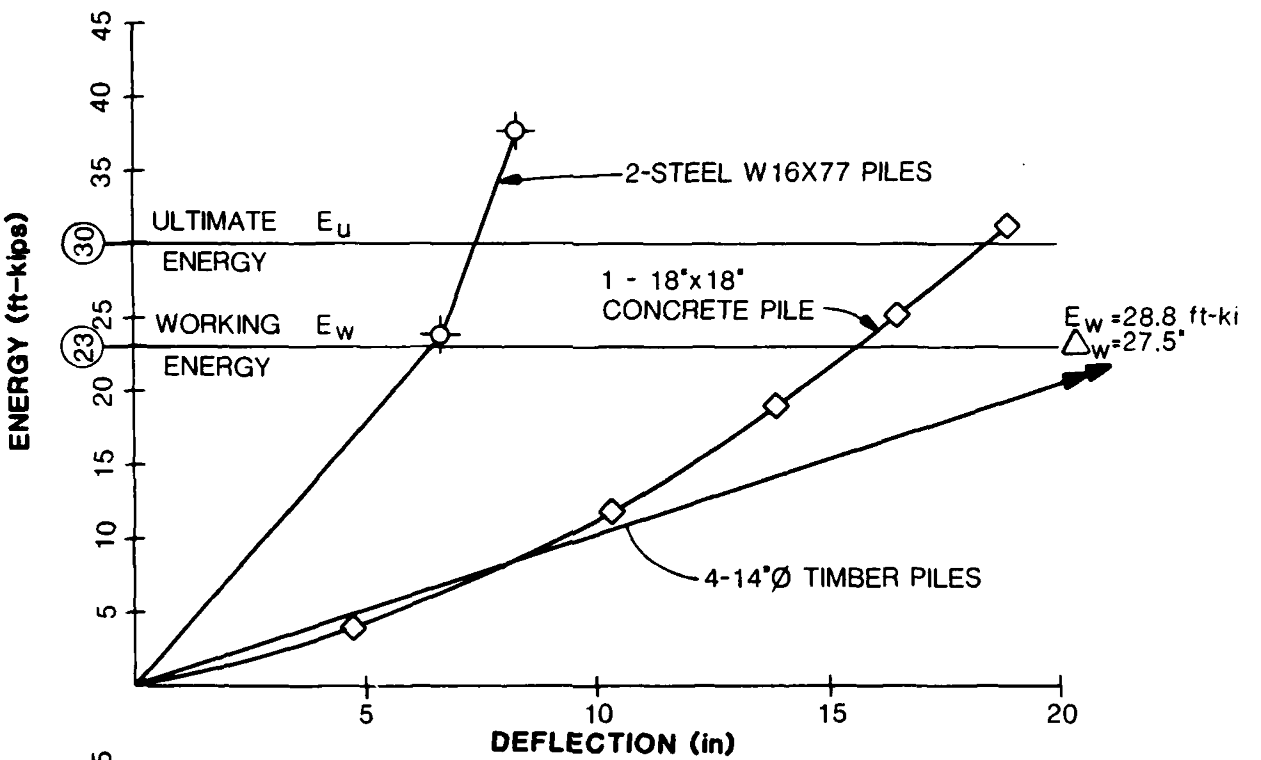
Figure 5.2. R-Δ Behavior for Concrete, Steel, and Timber Piles

5.3 TIMBER PILES

Timber piles of equivalent energy-absorbing capacity were also selected using a working stress approach. The allowable bending stress was chosen as 0.7 x f_r, where f_r is the modulus of rupture of the timber. For piling, the proportional limit is also approximately 0.7 times the modulus of rupture. This stress level is slightly higher than the allowable working stress typically used for timber design (with a 2.0 impact factor). The largest readily available pile has approximately a 14-in. butt diameter. The piling must be dual treated for maximum protection. Four 14-in.-diameter piles provide the required energy capacity. They provide a total working energy of 28.8 ft-kips.

5.4 ENERGY-ABSORBING CHARACTERISTICS OF CONCRETE, STEEL, AND TIMBER PILES

The reaction and energy versus displacement relationships for equivalent concrete, steel, and timber piles are shown in Figure 5.3. This figure is a good demonstration of the energy-absorbing characteristics of the various pile types. For a given energy level or reaction force, the concrete pile has a deflection approximately midway between the steel and timber piles. It is especially interesting to note that the concrete pile energy-absorbing characteristics closely resemble the equivalent timber piles for deflections less than 10 in. In addition, the stiffnesses (slope of the reaction versus deflection curve) of the concrete and timber piles are almost equivalent after the concrete section reaches its initial cracking stress. On the other hand, an equivalent steel pile is much stiffer than the prestressed concrete pile, with a correspondingly smaller deflection for a given energy input.



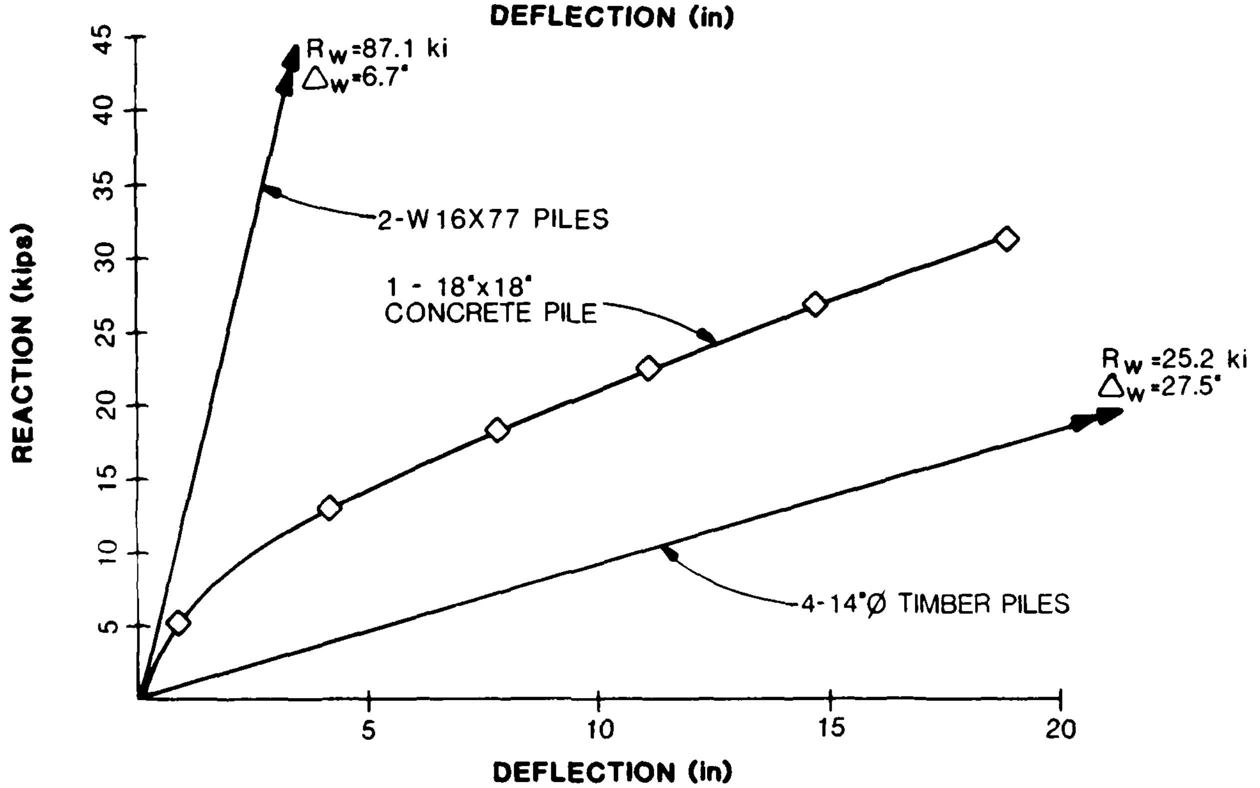


Figure 5.3. E-Δ and R-Δ Curves for Equivalent Piles

5.5 COST COMPARISON OF EQUIVALENT PILES

It is difficult to compare fender pile systems on an equitable basis because of the numerous factors involved in the design of a specific system. Our first step in comparing costs was to develop concrete, steel and timber fender piles with equivalent energy-absorbing capacities. The development of these equivalent systems was described in Sections 5.1 through 5.3, above.

The cost per ft-kip of energy absorbed (energy cost) for the various pile types is summarized in Table 5.1. As can be seen from this table, the energy cost for a prestressed concrete fender pile is much lower than the cost for an equivalent pile in either steel or timber. (See Figure 5.1 for equivalent piles.) Therefore, in a pile-for-pile comparison, the prestressed concrete piles appear to be more economical for a given energy input. However, the use of rubber fenders to absorb energy in a steel fender system and the fact that timber fender systems are often constructed without regard to specific energy requirements greaty influence the cost comparison of the various systems. The influence of these factors is discussed in further detail in Section 5.6.

5.6 COST COMPARISON OF SYSTEMS

As stated previously, the comparison of concrete, steel, and timber fender systems is extremely difficult. This is primarily due to the numerous variables involved in designing a specific system, the lack of consistent design requirements and methods for the various materials, and the personal preferences of the designer. To compare system costs, we have developed costs for each of the materials on the basis of installing individual fender piles at 10 ft on center. This is a reasonable assumption as bent lines on many Navy piers are spaced at 20 ft on center and fender piles are often placed at and halfway between bent lines.

TABLE 5.1
ENERGY COSTS FOR EQUIVALENT PILES

Pile Type	Impact Energy (ft-kips)	Number of Piles	Total Cost ($)	Energy Cost ($/ft-kip)
Concrete	23.0	1	2400	104
Steel	24.4	2	5280	216
Timber	28.8	4	3440	119

Notes

1. Impact energy is based on 70-ft-long fender piles (65-ft design span) loaded 15 ft from the top end.

2. Total cost is based on providing one 18- x 18-in. concrete pile, two W16x77 steel piles, or four 14-in.-diameter timber piles, including driving costs. Pile length is 70 ft.

3. Initial (first) costs only; replacement and life-cycle costs not included.

The concrete and timber fender pile systems use timber chocks and walers, while the steel fender pile system uses steel walers at the top of the pile and steel walers at a lower level to provide lateral support to the steel piles. Actual designs for steel fender pile systems may require more than two levels of walers to adequately brace the pile to meet design requirements. The top of the fender piles for both concrete and steel fender systems is supported by Uniroyal Model 25 Delta fenders or blocked solid against the pier structure. In the timber pile system, 8-in.-diameter x 1-ft-6-in.-long extruded rubber fenders are used behind each fender pile and midway between piles. This rubber fender was chosen as being representative of those used with timber systems. In addition, the steel system typically requires corrosion protection; therefore, cathodic protection has been included in its cost. The cost of log camels has not been included, as it would be common to all systems. Table 5.2 summarizes the energy costs for each of the systems.

The energy costs to provide a concrete system are approximately 30% to 35% less than for the steel system with a rubber fender and 40% to 45% less than the timber system. It is important to note that a rubber fender is required in the steel system to provide adequate energy absorption. Also, the timber system is unable to furnish adequate energy capacity.

The present value to install and rebuild the various systems described above pver a 25-year period is summarized in Table 5.3. As can be seen from the data in Table 5.3, the prestressed concrete fender piles are more cost-effective than either the steel or wood systems.

In summary, the prestressed concrete piles absorb more energy than a steel pile. The steel H-shaped pile is very susceptible to damage, when hit at an angle, due to local buckling of the flange. Thus, maintenance costs due to damage of concrete piles should be less than for a steel system of equal capacity. A timber fender pile

TABLE 5.2
ENERGY COSTS FOR CONCRETE, STEEL, AND TIMBER FENDER SYSTEMS

Fender System	Total Energy (ft-kips)	Maximum Reaction (kips)	System Cost per Pile ($)	Energy Cost per Pile ($/ft-kip)
Concrete with solid block	23.0	28.2	3280	143
Concrete with rubber fender	25.2	28.2	3950	157
Steel with solid block	12.2	50.4	4950	406
Steel with rubber fender	24.6	50.4	5620	228
Timber	7.7	6.3	1990	258

Notes

1. Energy costs are in dollars per pile for 10 ft of berth. Includes installation, rubber fenders or solid block, hardware, walers, and cathodic protection as appropriate.

2. Total energy and maximum reaction are defined as the total energy absorbed by the system and the maximum reaction against the ship, respectively, from a berthing impact 15 ft below the top of the pile.

TABLE 5.3
PRESENT VALUE FOR CONCRETE, STEEL, AND TIMBER FENDER SYSTEMS

Fender System	System Cost per Pile ($)	Expected Life (yrs)	Present Value ($)	Total Energy (ft-kips)	Present Energy Cost ($/ft-kip)
Concrete with solid block	3280	25	3470	23.0	151
Concrete with rubber fender	3950	25	4120	25.2	163
Steel with solid block	4950	25	4950	12.2	406
Steel with rubber fender	5620	25	5620	24.6	228
Timber	1990	8	2730	7.7	355

Notes

1. Value taken as 10% for economic analyses of Navy investment proposals, Economic Analysis Handbook, NAVFAC P-442, July 1980, Section 3D [5.2]. Inflation is included in the selection of this value.

2. Present value represents a present sum of money required to construct the fender systems and replace them as they wear out over a 25-year period. The timber chocks and walers will be replaced at the end of 15 years. The timber fender piles will be replaced at the end of 8 and 16 years.

3. Yearly maintenance costs due to damage from ship impact are not included.

system costs much less to install than either a prestressed concrete or steel fender system. However, the energy that it can absorb is significantly less. Therefore, the protection that it offers to the pier and the ship is much less than the other two systems. Thus, maintenance costs due to damage should be higher. It is apparent that a prestressed concrete fender pile system is a viable and economical alternative to be considered for installation at Navy piers.

5.7 FOAM FENDERS SUPPORTED BY PRESTRESSED CONCRETE PILES

In these concepts, the prestressed concrete piles are used as reaction piles for the foam-filled fenders. Figures 5.4, 5.5, and 5.6 were provided to ABAM by NCEL as possible systems utilizing prestressed concrete piles. The energy capacity of these systems is not known, nor whether the full working energy capacity of the foam fender can be developed before the concrete piles reach their capacity. Our task was to develop a comparable system utilizing the prestressed concrete fender piles developed in this study for cost comparison with the other three systems.

In developing our concept (see Figure 5.7), a 12-ft-long x 6-ft-diameter foam-filled fender was used. Its estimated ultimate energy capacity, at approximately 64% compression, is 273 ft-kips. At extreme high tide, the foam fender reaction would be near the top of the piles so that all or a majority of the impacting energy would be absorbed by the foam fender, and very little by the concrete piles. It was assumed that the concrete fenders must fully support the foam fender or, as a minimum, the system comprised of both foam and concrete fenders must be capable of supplying a minimum of 273 ft-kips without damaging the concrete piles. At low tide, it was assumed that the foam fender would react on the piles at a distance of 15 ft below the top support. This load point is critical for the selection of the number of concrete piles required to support a foam fender reaction of 195 kips.

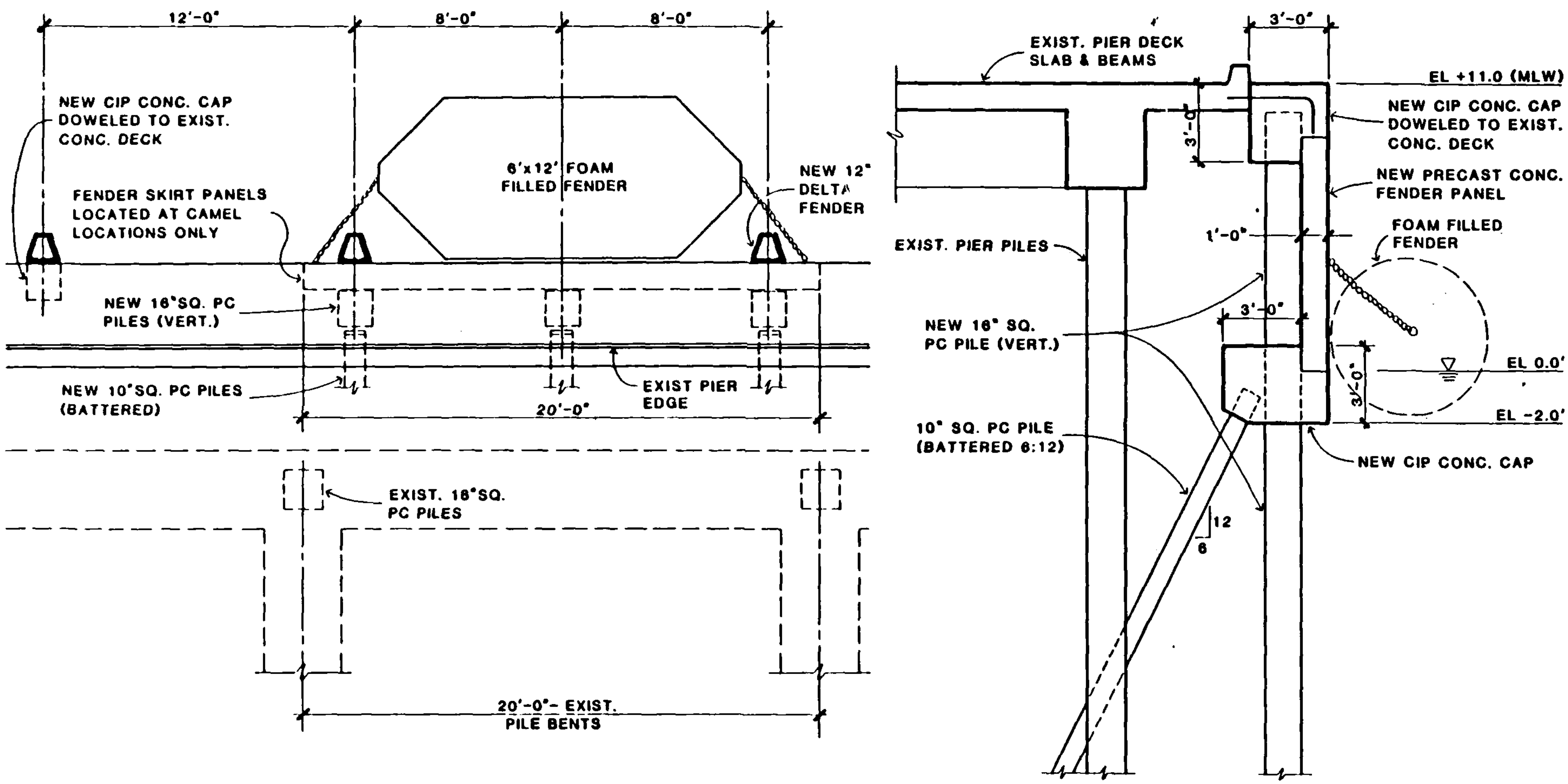


Figure 5.4. Concrete Fender System Using Concrete Battered Piles

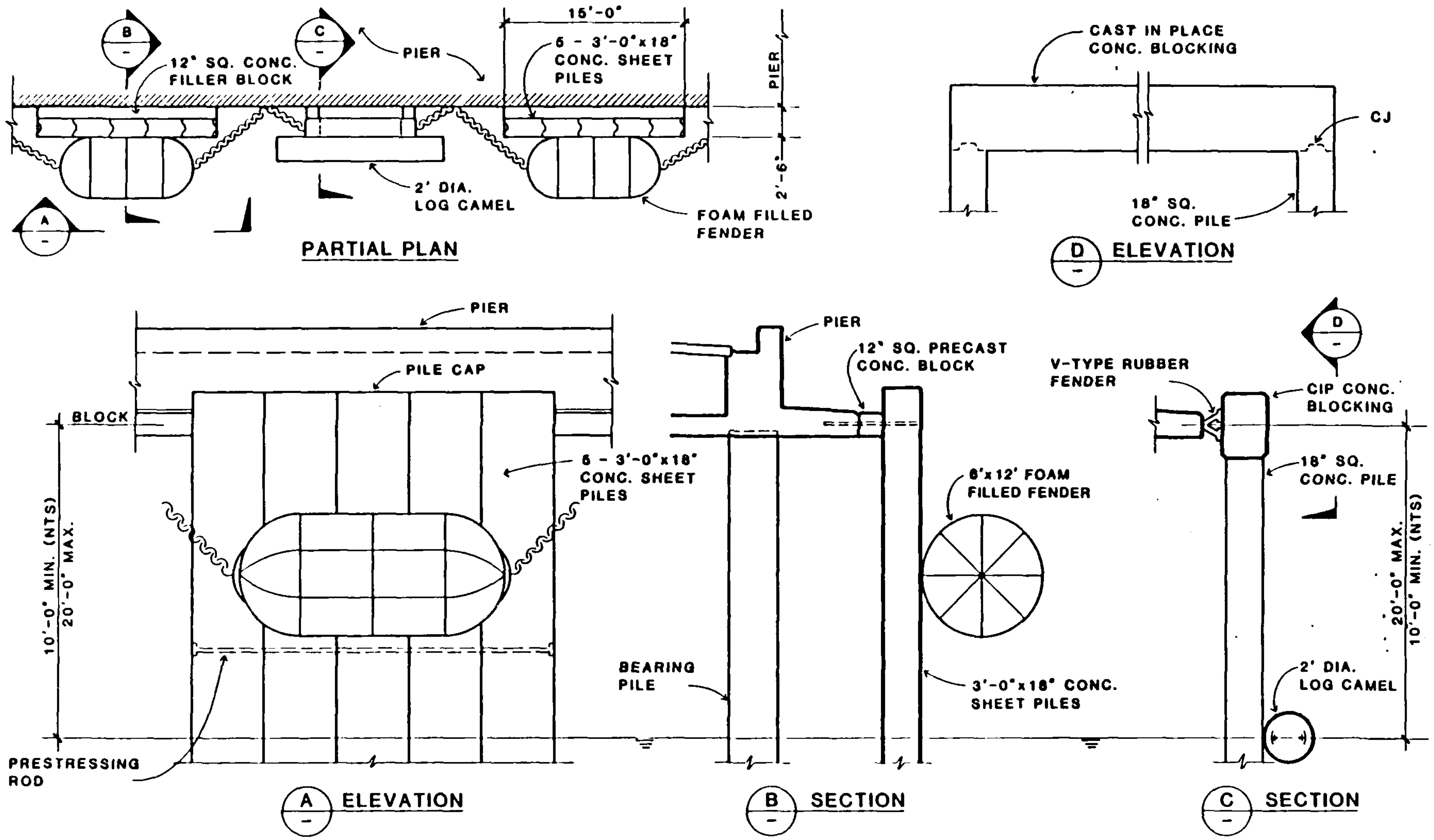


Figure 5.5. Concrete Fender System Using Concrete Sheet Piles

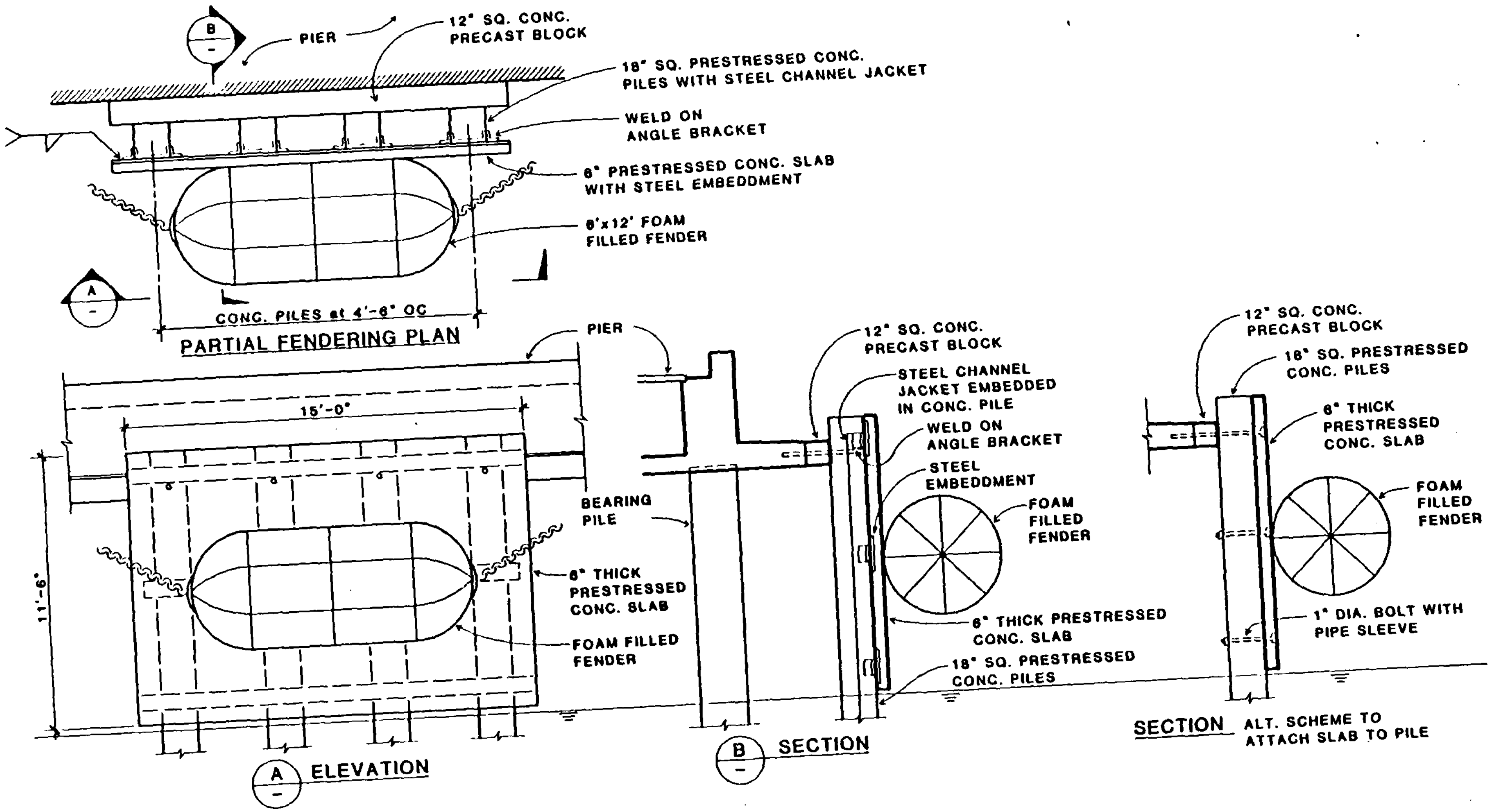


Figure 5.6. Concrete Fender System Using Concrete Bearing Panel

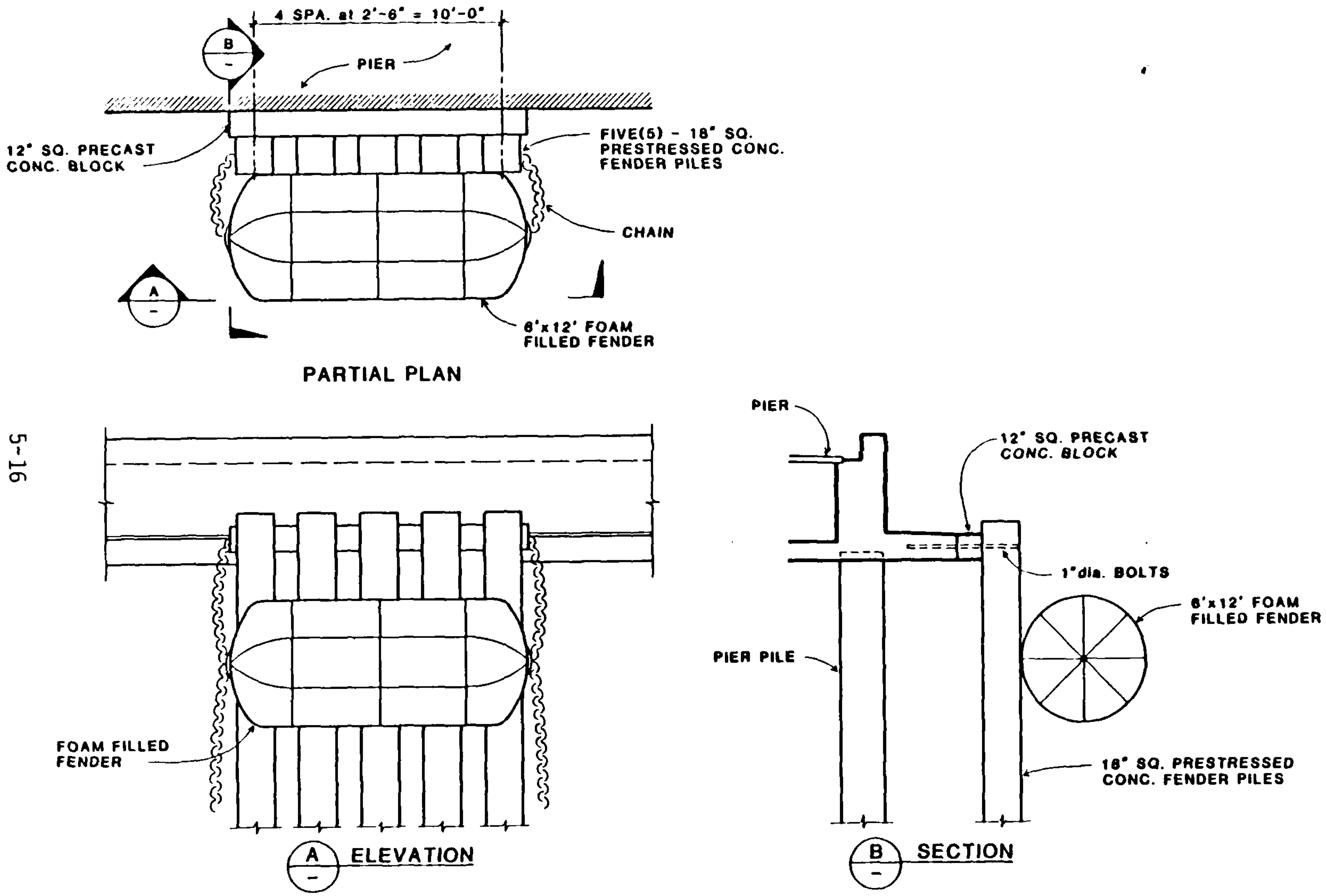


Figure 5.7. Concrete Fender System Using Concrete Fender Piles Only

The pile stiffness is a function of the prestress level in the strand. Stiffness is defined as the load required to move the pile a unit displacement. The prestress level (60 or 150 ksi) affects the deflection that the pile can take but does not affect the reaction in the pile. Since the stiffer piles (150-ksi prestress level) do not absorb as much energy as the more flexible piles (60-ksi prestress level) for a given foam fender reaction, there is no advantage to using a stiff pile to support a foam fender. As a matter of fact, flexible piles will add to the energy-absorbing capacity of the foam fender system, which would allow the use of a smaller, less costly foam fender.

5.8 FOAM FENDER PANEL COSTS

The estimated construction costs of the various foam fender support panels shown in Figures 5.4 through 5.7 are summarized in Table 5.4. The cost estimates shown in Table 5.4 are based on conceptual sketches. Therefore, the estimates only indicate the relative cost among the various options. Additional design work would be required to produce a more accurate estimate of the actual costs for each of the fender panel options.

TABLE 5.4
FOAM FENDER PANEL COSTS

Foam Fender System	Estimated Cost ($/panel)	Relative Cost Factor
Figure 5.4 with batter piles	$39,578	1.58
Figure 5.5 with sheet piles	$38,385	1.53
Figure 5.6 with bearing panels	$27,916	1.11
Figure 5.7 with bearing piles	$25,096	1.00

Note: Includes the cost of a foam fender

SECTION 6
DURABILITY OF PRESTRESSED CONCRETE PILES

6.1 DURABILITY OF CONCRETE PILING IN SERVICE

Documentation of the condition of prestressed piling in service is lacking. This supports the opinion of many engineers involved in the design and manufacture of piling that the performance of prestressed concrete exposed to seawater is excellent, especially when one considers the aggressive nature of the physical environment. The absence of significant reporting of corrosion, spalling, failures, or other evidence of widespread problems is not simply an indication of lack of inspection, but in most cases only a lack of distress.

Inspection of piling is often made at the time of new construction on, or in addition to, existing waterfront structures. Our knowledge of these inspections supports the good performance record. Likewise, inspection of structures damaged by ship impact or other structural overloads has revealed distress only on those piles directly impacted and often these are repairable. Successful repair of piling damaged during driving has been accomplished and most remain durable, if the repair is performed properly.

A private communication with the Florida Department of Transportation (FDOT) Bridge Inspection Department [6.1] revealed some concern on their part following recent condition surveys (conducted in the past year) of prestressed concrete piles in service. Many pile-supported bridge piers were constructed for the Florida highway system in the 1960s, and many of the square piles are badly spalled at the corners due to corrosion of prestressing strand. Cover over the strand was generally 2-1/4 in. This distress is occurring just

above high water level in the splash zone. Their surveys show that the onset of corrosion is 15 to 20 years after construction with a small number of samples showing corrosion in 6 to 10 years. No structural failures have occurred, but they have begun a maintenance program to repair and arrest the damage. In addition, the FDOT has changed their concrete specification to require a lower water/cement ratio, pozzolanic additions and, in some cases, low heat of hydration (Type II) cement. Florida limerock aggregate has a low density and high absorption, but it is thought that by improving the cement matrix, they can improve the durability of new piling.

6.2 FACTORS AFFECTING CORROSION

6.2.1 Quality of Concrete

Sufficient technology and materials exist to provide concrete which will resist even the most severe seawater exposure without the addition of a protective system. The designer and manufacturer should include most or all of the following elements in prestressed concrete piling:

- Low water/cement ratio
- Addition of a mineral admixture
- Cement with proper tricalcium aluminate (C_3A)
- Small maximum size of coarse aggregate
- High energy mixer
- Adequate placing and vibration procedures
- Control of curing temperatures
- Good air-void system

Although these elements for good quality concrete in seawater are listed somewhat in order of their importance, they must all be combined to take advantage of their synergistic effect.

a. Water/Cement Ratio

Maximum ratios from 0.40 to 0.45 for concrete exposed to seawater exist in the various codes, with 0.40 the most commonly specified value in recent years. Whereas this low water requirement was achievable by only a few producers years ago, it is now a practical limit for any prestressed concrete producer. The use of high range water reducers (HRWRs), sometimes called superplasticizers, has enabled a 15% to 30% water reduction as compared to concretes made prior to the mid-1970s. In fact, some evidence exists that might soon place a limit on the minimum water/cement ratio to be used. It is desirable to have sufficient free water available to react with unhydrated cement which is always present in large quantities in concrete with high cement factors. It is now possible to practically make concrete with a water/cement ratio of 0.28 to 0.32 and properly consolidate it. Some researchers suggest this may be too low for good durability and that 0.36 to 0.38 may be an optimum target.

b. Mineral Admixtures

Fly ash from coal-burning utilities, finely ground slag from steel manufacturing, and silica fume from silicon metals manufacturing are reactive with cement hydration products and are known to improve the durability of concrete in seawater. It has been known for a long time that these pozzolanic materials increase resistance to sulfates, but recent research has revealed additional benefits available from silica fume additions to concrete. These tests, aimed toward resisting the penetration of deicing salts into bridge decks and parking garage floors, have resulted in an increase in electrical resistivity of several hundredfold.

These admixtures also help to prevent deleterious expansion from alkali-silica reactions or combined sulfate/alkali-silica attack. The optimum addition rates to concrete vary, depending on the cement properties and the type of mineral admixture used, but it is generally in the 10% to 25% range.

c. Water-Reducing Admixtures

A new generation of HRWRs has been or is being introduced which will retain their slump for 30 to 60 minutes. This development has mitigated the slump-loss or pot-life problem which, although it didn't affect precasters greatly, was a drawback to their use in prestressed piling. Conventional water reducers have also been combined with HRWRs to greatly reduce the high water demand of these rich mixes. The precast concrete manufacturer must select a water reducer and its dosage rate carefully to optimize the water reduction and workability but still improve the early strength gain.

d. Portland Cement

In past years, specifications for seawater exposure reflected the results of research on cement composition which stated that the cement should be a Type II or Type V with a C_3A content below 7% for good sulfate resistance. Type V cement though, should not be used since it does not perform well in seawater. Recent testing and analysis show that a low water/cement ratio is more important for protection from sulfates in the seawater and that the C_3A content should be higher to help concrete resist chloride depassivation of reinforcing and prestressing steel. The optimum amount of C_3A is 6% to 10%, which is now the stated limit in Corps of Engineers specifications and others.

Prestressed concrete requires high strength for detensioning which must be achieved in 16 to 40 hours after casting for efficient production. This usually means that the precaster uses Type III, high early strength cement, or normal Type I cement which has been ground somewhat finer. Some specificers do not permit Type III cement, and even Type I in some cases, for concrete in seawater because of supposed higher shrinkage and heat generation. It has been shown that the high tensile strength gain of Type III offsets any increased shrinkage. Heat generation may be a problem only in mass concrete, over 24 in. thick. Since little, if any, piling is manufactured with a thickness of over 24 in., it is detrimental to economical production of prestressed piling to limit the choice of cement to Type II provided criteria stated above for C_3A content are adhered to.

e. Coarse Aggregate Size

Guidelines for sea structures now suggest that the cover be at least twice the maximum size of coarse aggregate and that the aggregate size be limited to the smallest practical. This is to limit the amount of microcracking in the concrete due to restraint of paste shrinkage and to ensure good flow of concrete between the form and steel reinforcement. The net result of following this recommendation is a more impermeable cover over the reinforcement.

Small size of coarse aggregate is also desirable for high strength concrete. The smaller size of aggregate produces more surface area, which in turn results in increased paste to aggregate bond and higher compressive strength. This is not so for leaner concretes because there is insufficient paste to coat the larger surface area; thus, larger coarse aggregate size yields higher compressive strength in that case. The

optimum aggregate size for most high strength concrete is found to be 3/8 to 5/8 in.

f. Concrete Mixer

The tendency for high strength concrete to be sticky and generate heat rapidly makes it beneficial to use high energy mixers. The objective is to mix the concrete in the shortest time possible while still achieving a uniform distribution of materials and proper air entrainment. The pan or turbine type of mixer is most commonly used; however, some new mixers are now on the market which utilize other means of achieving a short, intensive mixing action. An example of one of these is the Nikko mixer, manufactured in Japan.

g. Concrete Placement and Vibration

It is anticipated that prestressed concrete fender piles will be fabricated with high strength concrete containing 650 to 850 lb/cu yd of cement, 75 to 200 lb/cu yd of a mineral admixture, and a high dosage of water-reducing admixture. These mixtures will be less workable than ordinary concrete and this factor must be considered in the placement and vibration of the concrete. Prestressers work with rich mixes similar to this on a daily basis so the cohesive properties of such concrete are not new to them. However, not all of them are equipped to handle and place the concrete without adding excessive water for ease of placement. Likewise, the concrete must be consolidated with internal or external vibrators designed to obtain maximum density of the finished product.

It is not common to use external vibrators on prestressed piling since the configuration of the formwork in the long beds is not conducive to mounting vibrators on the form. The close spacing of prestressing strands and reinforcing ties or spirals

limits the size of internal vibrators which can be used, thus increasing the need to select a vibrator and vibrator spacing which properly consolidate the concrete. Round or square vibrators with a maximum cross-sectional dimension of 1-1/4 to 1-1/2 in. and a frequency of 7,000 to 11,000 vibrations per minute are found to be adequate. They should be inserted vertically in the fresh concrete on about 6-in. centers. It is difficult to overvibrate these cohesive mixtures and the tendency is to undervibrate. The time of insertion is not easy to state, but must be determined in practice from observations and density measurements. A range of time of vibration can be made in the laboratory at the time of designing the mix or confirming the application of an existing mix design.

h. Accelerated Curing

Some cracking in prestressed piling may occur from excessive temperature differentials within the concrete. This is isolated to cases where elevated curing temperatures are produced by steam or other energy sources and the concrete is suddenly subjected to cold temperatures after the forms are stripped. The piling should not be exposed to an ambient temperature more than 40°F lower than the concrete temperature. This can be controlled by allowing curing covers to remain in place after the heat is shut down, as is most often done in the industry today.

i. Air-Void System

It is mandatory that concrete piles exposed to freezing in the tidal and splash zones have a properly entrained air system for good freeze/thaw durability. Concrete used in these fender piles with a low water/cement ratio and low permeability will not require the normal 4% to 8% of entrained air to achieve good durability in the mortar matrix. Research has shown that

3% to 5% volume of air in the fresh concrete will provide excellent freeze/thaw resistance. Piling exposures involving a large number of freeze/thaw cycles should require that hardened concrete samples be examined for a proper air-void structure as defined by ACI. The air-void system should be verified at appropriate intervals during piling production.

6.2.2 Concrete Cover

The thickness of cover over the prestressing steel has been a controversial issue among researchers and specifying agencies. Tests and studies related to corrosion of prestressing steel have shown that a cover thickness of 1-1/2 in. will provide good protection against chloride attack when high quality concrete made with small aggregate size is used. Some authorities feel that at least 60 mm (2-3/8 in.) of cover is necessary for cast-in-place concrete and something less may be acceptable for plant-cast, prestressed concrete. It is important to remember that durability is enhanced by minimizing microcracking in the cover. Microcracking is reduced by using small size coarse aggregate and high quality paste, and providing proper curing.

Few penetrations through the cover in prestressed piling are ordinarily found, but one of these, lifting loops, is sometimes found to be a contributor to corrosion. The loops provided for handling the piling prior to driving are commonly made from short lengths of prestressing strand and are burned off with a cutting torch and patched prior to driving. The area around the strand penetrations should be recessed at the time of finishing the fresh concrete to provide for a thicker patch. The patch can be done effectively with epoxy mortar, but the choice of epoxy resin and method of application should consider ambient temperature and moisture.

6.2.3 Exposure in the Tidal and Atmospheric Zones

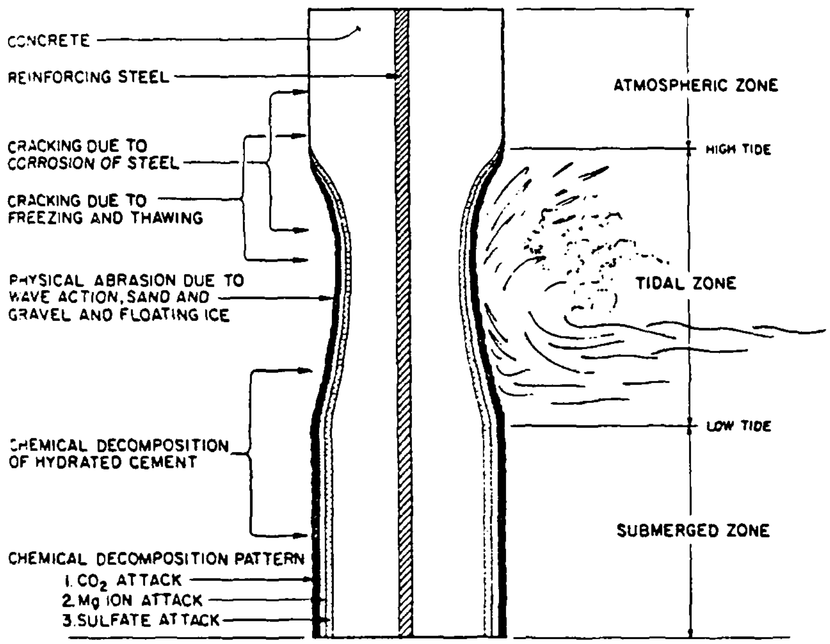
Most damage to concrete piling occurs above the mean low water level where it is exposed to nearly all of the physical and chemical processes of deterioration. This is best illustrated by Mehta's schematic diagram shown in Figure 6.1 [6.2]. Cracking due to steel corrosion and freezing and thawing is localized in the tidal zone. Corrosion from oxygen concentration cell attack in this area is caused by large differences in oxygen content over short distances. Freeze/thaw damage may occur where the concrete remains saturated in the splash zone. Corrosion in the submerged zone is rare because of the uniform low oxygen content below the low tide level. In addition to the cracking sources shown in this figure, another likely cause for cracking is from impact loads of berthing ships.

Regardless of the cause of cracks or spalls in prestressed fender piles, corrosion of the prestressing strand is the most likely cause for failure and should be monitored and prevented or arrested.

6.2.4 Cracks and Corrosion in Concrete

There is general agreement that crack width and size of reinforcing steel have little influence on the rate of unacceptable corrosion in structures [6.3, 6.4, 6.5]. Cracks narrower than about 0.012 in. will seldom result in corrosion, while cracks wider than this threshold value will produce only localized rust. Chloride penetration at a crack will depassivate the steel; likewise, depassivation will be accelerated by reducing alkalinity of concrete around the crack. Corrosion due to this localized deterioration can be reduced or eliminated when high quality concrete is used. In any case, the extent of depassivation is limited to a very short distance from the crack location, less than three-strand or bar diameters.

The more important considerations are orientation of the crack to the steel, number of cracks in the vicinity, and activity of the



From: P. Kumar Meha. Performance of Concrete in Marine Environment, Publication SP-65, American Concrete Institute

Figure 6.1. Deterioration of a Concrete Structure in Seawater

crack. Cracks following the length of strand or rebar are far more serious and should be prevented. Corrosion products are likely to spall concrete cover over the steel and lead to widespread deterioration at longitudinal cracks. Longitudinal cracks are unlikely to occur in prestressed fender piles unless created by plastic shrinkage. This possibility is further reduced because it occurs from too rapid drying of an exposed surface while the concrete is still plastic, and most producers cover the piles before this could happen.

A condition where multiple transverse cracks occur in the tidal or splash zone is a more possible cause for concern. In this case, the very high resistivity of a high quality concrete will be negated by the formation of a corrosion cell between the anodes and cathodes formed at adjacent cracks. Current flows from one crack to another through the seawater, which has a low resistivity. The degree of severity of this condition has not been quantified because little research has been performed on multiple-cracked specimens. Our experience indicates that single transverse cracks do not lead to corrosion with high quality concrete. It is our recommendation that NCEL investigate portions of the test piles to determine the potential for corrosion due to multiple cracks.

Narrow cracks which remain open in a seawater environment are likely to heal autogenously or by accumulation of mineral deposits [6.2, 6.6]. Cracks which are actively opening and closing or, more important, progressively widening, are more likely to be a problem. This would only be likely in a case where a pier is constantly subjected to high impact overloads.

The risk of serious corrosion in marine structures due to cracking is increased if extensive microcracks are present and are made continuous from external sources. However, there is almost no likelihood of this occurring when all the factors mentioned for high quality concrete are incorporated into the design and fabrication of the piling.

Prestressing strand must be isolated electrically from any inserts embedded in concrete to protect it from galvanic corrosion between dissimilar metals.

6.3 PROTECTIVE SYSTEMS

The employment of high quality concrete will protect embedded steel and/or slow or prevent chloride penetration into the concrete. If additional protection is deemed necessary to protect open cracks or to offset the impact of corrosion due to multiple cracks, the most effective means of corrosion protection appears to be the use of epoxy-coated prestressing strand. In fact, NCEL currently has a test installation near Norfolk, Virginia, which has epoxy-coated strand in the piling. A Florida Department of Transportation bridge near Melbourne, Florida, currently under construction, utilizes this means of protection in the fender piles protecting the bridge piers. The epoxy coating is a positive method of decreasing or preventing cathode areas on the strand. The possibility of corrosion is eliminated whenever the coating remains intact at a location where the chloride content is sufficient to depassivate the steel. Also, if the coating is damaged at a site where depassivation occurs, the intact coating on adjacent areas reduces the rate of attack by reducing the effective cathodic area. The system seems to be even more effective than epoxy-coated rebar in that the coating thickness is greater on the strand and the possibility of a holiday in the coating is greatly reduced. The reason for the latter advantage is that the strand is not bent or exposed to as much abrasion during fabrication as rebar.

There are some drawbacks to using epoxy-coated strand in fender piles, but they should diminish as use of epoxy-coated strand increases. The initial cost is high, approximately twice the cost of uncoated strand, and the product is currently available from only one domestic supplier. Some problems have been encountered by prestressed concrete manufacturers when stressing the strand.

Special strand chucks and/or jacking methods must be used to successfully perform this operation; however, as familiarity with this process increases, the problem will be eliminated. There is a maximum temperature limitation of about 140°F which must be observed during the concrete curing phase that some manufacturers would normally exceed, but in reality is not overly restrictive. This limit exists to prevent excessive creep deflection of the epoxy when the strands are detensioned. Also, if epoxy-coated strands are to be used, the effects of spalling due to impact loads need to be investigated.

A study was made to determine how much could be spent on additional corrosion protection for a pile to justify the increase in the pile's life. If the economic life of the pile without epoxy-coated strands was chosen to be only 12.5 years, but 25 years with epoxy-coated strands, an initial cost increase of 31% is all that could be justified. Current cost data indicate a pile with epoxy-coated strands would cost 35% to 50% more than one without. Based on our engineering judgment and experience, we feel that a 25-year life expectancy for a prestressed concrete fender pile without epoxy-coated strands is reasonable. However, the actual life expectancy remains unknown. As we already anticipate a life expectancy in excess of 12.5 years, the extra cost does not appear to justify the use of epoxy-coated strands.

There are some effective concrete coatings which could be applied on the piling above the submerged zone, i.e., concrete in the tidal and splash zones. These coatings are epoxy resin or coal tar epoxy. It is perceived that a coating might be used to reduce the permeability of concrete which was or could not be made to the quality standards advocated herein. Coatings are an effective method of ensuring continuity of protection when applied over and around the perimeter of patches on repaired piling. They are not effective as a means of stopping corrosion after it has begun; in fact, that application serves to mask further corrosion when it could otherwise be detected

and arrested or repaired by other procedures. It is important to have the coated surface as free of holidays as possible since chlorides become more concentrated in discontinuous areas. Recent tests described in National Cooperative Research Program (NCHRP) Report No. 244, "Protective Coatings for Concrete" [6.7]. have shown that there are at least two or three epoxy or silane sealers which provide effective barriers to chloride penetration. The major drawback to these systems is the cracking of the pile under impact loading. This will also crack the coating and significantly reduce its effectiveness.

There is at least one corrosion inhibitor available now which can be added into a concrete mixture to provide some protection. Tests [6.8] have shown that calcium nitrite will prevent chloride depassivation of steel until the amount of chlorides in the concrete reaches a certain threshold ratio of one half (by molecular weight) the amount of available inhibitor. Thus, where it could be really beneficial in porous concrete, its effectiveness is negated sooner because the movement of chlorides into the concrete is faster. Other NCEL tests have indicated that the primary role of the calcium nitrite is to decrease permeability. This very high cost material would have limited value in high quality concrete.

The use of cathodic protection (CP) systems for corrosion protection is increasing rapidly for concrete bridges exposed to deicing salts and concrete marine structures. Most of the proprietary bridge deck protection systems use high voltage and require a large amount of maintenance and, therefore, are not thought to be cost-effective for application on new structures. However, CP systems with galvanic, anodes can be effectively applied on existing piling where some corrosion has been detected, but will still be serviceable if arrested. This is particularly true where corrosion is in a tidal or submerged zone.

Repair of severely damaged piling can be effectively made by applying jackets and filling the space between the prepared concrete surface and the jacket with concrete or high quality grout.

6.4 RECOMMENDATIONS FOR DURABILITY

6.4.1 Serviceability

It appears that corrosion will not be a problem during the life of a prestressed concrete pile as the transverse cracks will only be open for short periods of time and some piles will have short lives due to loads which exceed the design loads. Rather than providing expensive reinforcing coatings, pile coatings, or concrete additives to increase the life expectancy of the concrete fender piles, it is probably more cost-effective to monitor the corrosion and add inexpensive cathodic protection, if and when needed.

The design life and serviceability requirements for prestressed concrete fender piles should be established following the final testing phase. Piling incorporating the concrete mix design and manufacturing quality standards discussed earlier should have a service life of 25 to 50 years under normal use, with little or no maintenance required. Normal use is defined here as piling that may be cracked transversely due to ship impact. The crack(s) may open and close with subsequent berthings, but the crack will always close when unloaded to a narrow width (less than 0.012 in.). It has been seen that, in the case of piles made with no protective system and using permeable concrete (as in the case of the Florida bridge piers), corrosion damage requiring repair or replacement can be expected in 15 to 20 years. Thus, the worst-case scenario will still produce prestressed concrete piles with a life almost double that of wood piles. In this case, the life could be expected to be greatly increased by using epoxy-coated strand or passive cathodic protection or, at a minimum, providing a protective coating on the

concrete above the low water level, but these measures would not be cost-effective.

With these factors in mind, we conclude that the fender piles should be manufactured from high quality concrete used by most of today's fabricators and monitored for corrosion somehow (see below); inexpensive passive anode corrosion protection should be added, if and when needed.

6.4.2 Condition Monitor

It is believed that a low-cost system can be provided which will measure the rate of corrosion or galvanic current flow in piling to give an indication when repairs or protective systems need to be applied. NCEL has a laboratory-proven system which measures current flow normal to the concrete surface. It is strongly recommended that this system be developed for field use since no other method for measuring the rate of corrosion exists.

If such monitoring can be provided, then low-cost passive cathodic protection systems can be added, if required, to prevent further corrosion damage.

6.4.3 Condition Surveys

The condition of prestressed concrete fender piles should be assessed periodically by visual inspection and simple nondestructive testing. These inspections would not be costly since they are concentrated on the piling exposed above the low water level and could be performed with little or no diving. The condition surveys should be performed on a routine basis every five years and after pier damage or high overloads have occurred.

SECTION 7
PILE DUCTILITY

The high ductility exhibited by the prestressed concrete fender piles considered in this study allows them to provide high energy absorption and low reaction forces during ship berthing, and thus it is desirable to maximize the ductility of the piles to the greatest extent possible. Several factors limit the ultimate ductility of the piles including the degree of confinement on the concrete and the serviceability of the piles after having sustained very high compressive loads; however, the most important factor is spalling of the concrete cover of the pile. Even after the cover spalls, significant strength and deflection capacity remain, indicating that if the spalling could be delayed or minimized, the performance of the piles could be significantly increased.

Possible methods for enhancing the ductile performance of the piles include

- The use of steel fiber reinforced concrete

- The use of either internal or external confinement on the compressive concrete

- The reduction of cover to minimize the amount of spalling concrete

- The incorporation of narrow slots cut into the compressive face of the concrete in order to move the zone of maximum compressive strain inward to the confined region. The slots would be backfilled with a visco-elastic material suitable for the prevention of corrosion.

7.1 STEEL FIBER REINFORCED CONCRETE

The use of steel fiber reinforced concrete would most likely prove very effective in delaying or minimizing the spalling of the concret e cover on the compressive face of the piles due to the inherent high ductility of concrete with 1% to 2% steel fibers by volume of the concrete. In addition to preventing spalling of the cover, steel fibers can also somewhat increase the strength and, more particularly, the peak strain capacity of the concrete, both of which are key factors in determining the energy-absorption capacity of the piles. Other benefits of the fibers are that they would minimize crack width and thus would enhance serviceability after the pile had sustained high level loading, and also that they would provide confinement on the core concrete and thus would permit some reduction in the use of spiral reinforcement.

One problem with the use of steel fibers is their adverse effect on the workability of the concrete mix, and higher addition rates of steel fibers result in progressively worse workability. Factors affecting workability include fiber volume and aspect ratio (ratio of length to equivalent diameter), cement content, coarse aggregate size and gradation, water/cement ratio, and mixing methods. The use of flat, relatively wide (ribbon-shaped) fibers such as those produced by the Ribbon Technology Corporation of the U.S.A., or Harex Stahlfasertechnik GmbH & Co. KG of West Germany, can greatly enhance the workability of steel fiber reinforced concrete. Recommended mix proportions are discussed in ACI 544.1R, Chapter 3 [7.1].

7.2 CONFINEMENT

The use of either external or internal confinement in the concrete enhances ductility by both controlling shear stresses and providing extra strain capacity to the concrete. Internal spirals in the piles act much like conventional stirrups in structural beams, thus serving to control shear stresses. Furthermore, the use of heavy

spirals, much like the basketing effect of the high percentages of stirrups used in earthquake design, serve to control bursting stresses, minimize crack width, and provide for a plastic reserve once the cover spalls.

The use of external confinement (such as the use of a grouted steel pipe encasing the pile in the zone of maximum bending moment) would serve all the same functions as the internal spiral plus preventing or delaying spalling of the concrete cover in compression. The use of an external steel sleeve would also have the benefits of increasing the pile cross section in the zone of maximum moment, thus increasing load capacity. The external sleeve could also reduce the potential of corrosion of the internal steel after the pile had sustained high compressive loading, increasing the service life of the pile.

The use of external confinement, however, may affect the serviceability life of the pile. After loading to extremely high energy levels, the pile could have a permanent "set" or "kink" after removal of the load. Presumably, such a steel sleeve would be added after the prestressed piles were fabricated, and would require some dowel shear connectors to the precast piles. However, if properly designed, such steel sleeves could simplify the attachment of rub strips.

The cost of adding the external sleeve would increase the overall cost of the fender system. We therefore believe the use of an external confinement sleeve is not warranted for this application of service.

7.3 CONCRETE COVER

Another potential method to minimize the effect of the loss of the concrete cover is to reduce the clear cover over the reinforcing steel. The critical question here is how much the cover can be reduced without risking corrosion damage to the steel. Conventional

standards for cover were developed for conventional concretes and conventional reinforcing configurations; however the use of

- o High strength concrete with low water/cement ratios, small maximum size of aggregate, and microsilica fume will markedly reduce the migration of moisture, chloride ions, and oxygen through the concrete, thus reducing the corrosion potential.

- o Heavy steel reinforcing will minimize crack width and thus would also reduce the corrosion potential.

- o Epoxy-coated steel would also reduce the potential for corrosion and thus permit a reduction in cover.

7.4 NARROW SLOTS IN COMPRESSION FACE

Another method for enhancing ductility is the incorporation of narrow slots cut into the compressive face of the concrete in order to move the zone of maximum compressive strain inward toward the confined region. The slots would be backfilled with a visco-elastic material suitable for the prevention of corrosion. This method has the advantage of being simple in that the slots could be saw cut after the piles are fabricated; however, it has the disadvantage that although load-bearing visco-elastic materials are available for back-filling the slots, the load-carrying capacity would be reduced. Therefore, this method would be most effective for piles with minimal cover on the compression face.

Testing of a slotted pile specimen in the next phase of the fender pile program will prove the advantage of this concept. A judgment on the effectiveness of slots cannot be made at this time.

SECTION 8
RUBBING STRIPS AND WALERS

Three materials and three attachment concepts (a total of six alternatives) for attaching rubbing strips to prestressed concrete piles were investigated. The materials investigated were ultra high molecular weight (UHMW) plastic, treated Douglas fir, and rubber. A cost comparison of the various alternatives is presented in Table 8.1. Recommended attachment details for the pile and waler to the wharf are presented in Figures 8.1 and 8.6.

8.1 UHMW PLASTIC

This plastic offers the following advantages:

- No sparking or electrical conductivity
- No rotting
- No need to paint
- Easy to replace
- Low coefficient of friction
- Resists bacteria and barnacle growth
- Available predrilled, preformed, and cut to size
- No cold embrittlement (good from -115°F to +200°F
- Superior abrasion resistance
  - 5 times better than 304 stainless steel
  - 6 times better than carbon steel
  - 12 times better than phosphor bronze

Alternatives using UHMW are shown in Figures 8.1 through 8.5. In Figures 8.1 and 8.2, the rubbing strip is bolted with galvanized machine bolts to a 1/2-in.-thick galvanized steel plate. This assembly is clamped to the pile with steel rods and a spreader bar.

AD-A169 516 PRESTRESSED CONCRETE FENDER PILES - ANALYSIS AND FINAL TEST PILE DETAILS(U) ABAM ENGINEERS INC FEDERAL WAY WA MAY 86 NCEL-CR-86.009 N62474-84-C-3140 2/2

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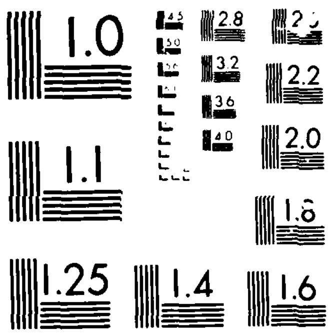

1.0
2.8
3.2
2.2
3.6
4.0
2.0
1.1
1.25
1.4
1.6

TABLE 8.1
RUBBING STRIP COMPARISON

Figure No.	Rubbing Strip Material	Initial Cost ($)[1]	Expected Life (years)	Cost per Year[3] ($/year)	Cost Ranking (per year)	Advantages	Disadvantages
8.1, 8.2	UHMW plastic	940	15	63	3	High durability	Relatively high cost
8.3	UHMW plastic	1,130	15	75	4	High durability	High cost
8.4	UHMW plastic	410	15	27	1	[2]High durability, lowest cost	
8.5	UHMW plastic	920	15	61	2	High durability	Relatively high cost
8.6	Treated Douglas fir	460	5	92	5		Short life, high life-cycle cost
8.7	Rubber	760	5	152	5		Most expensive

Notes: [1] Cost is for one 14-ft-long rubbing strip.

[2] This alternative has the lowest initial cost and lowest cost per year. Plastic will possibly last longer than 15 years and the bolt can be easily replaced.

[3] Without adjustment for interest income.

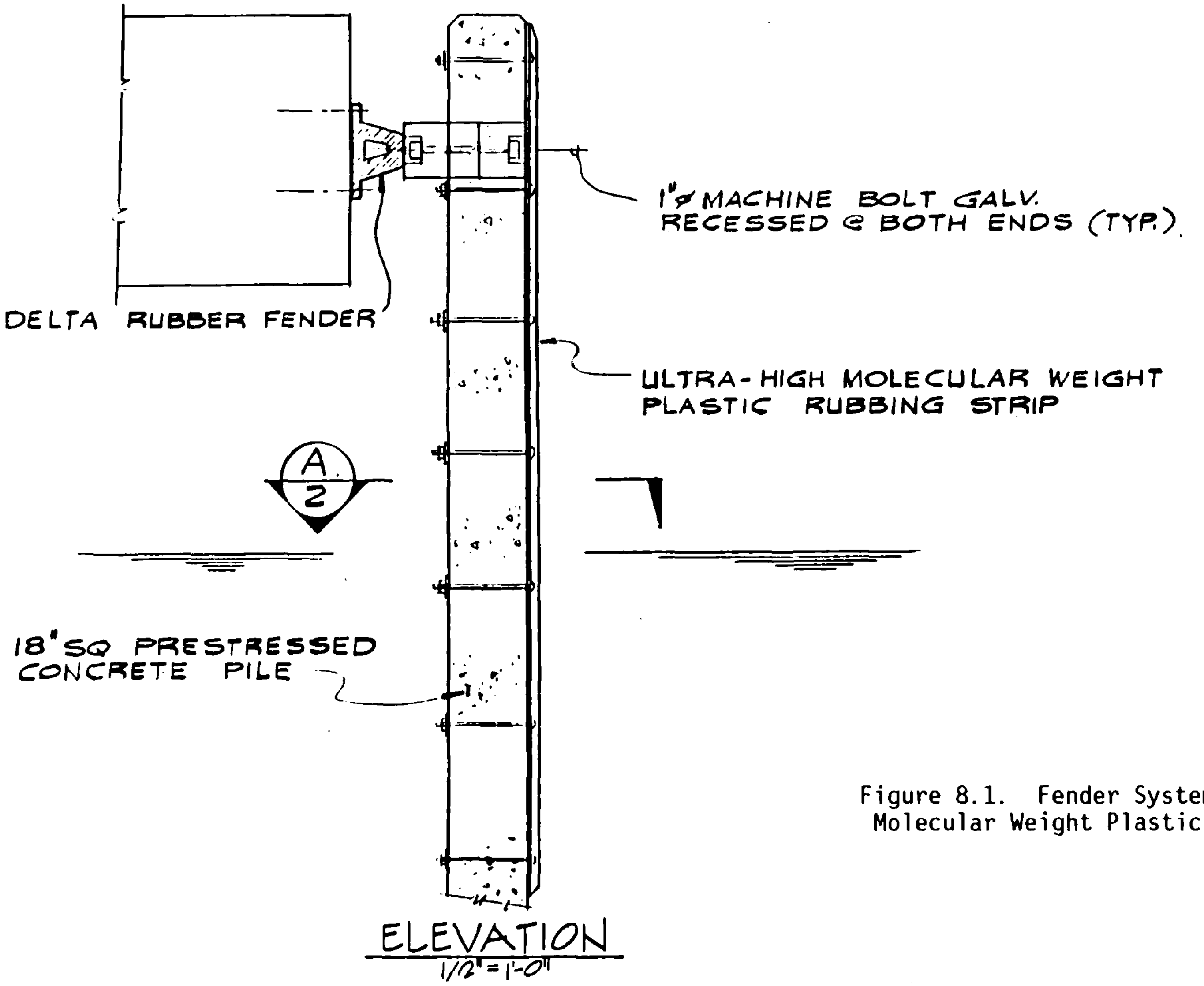


Figure 8.1. Fender System with Ultra High Molecular Weight Plastic Rubbing Strip

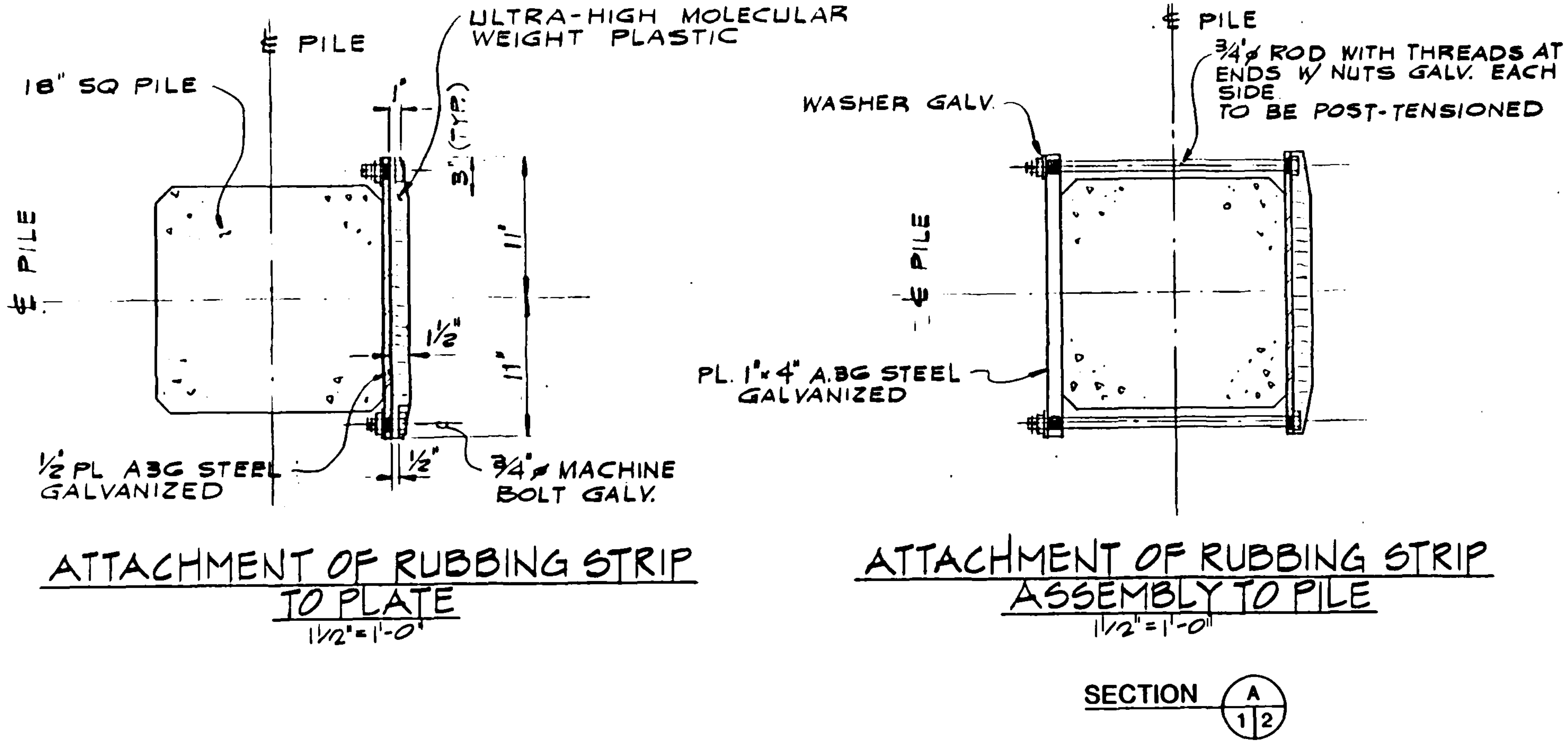
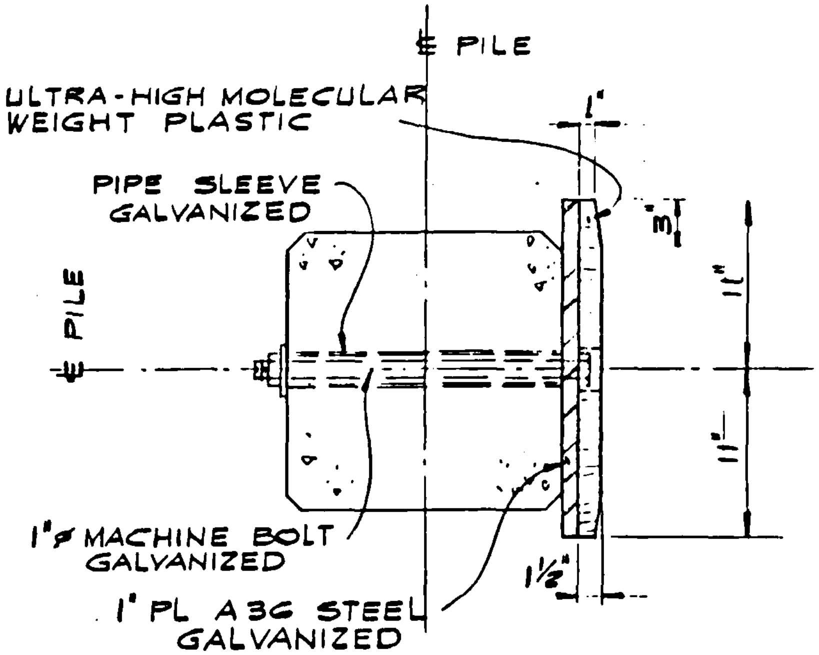


Figure 8.2 Fender System with Ultra High Molecular Weight Plastic Rubbing Strip



ATTACHMENT OF RUBBING STRIP ASSEMBLY TO PILE

1½" = 1'-0"

NOTE:
ATTACHMENT OF RUBBING STRIP TO PLATE SAME AS ON FIGURE 8-2

Figure 8.3. Fender System with Ultra High Molecular Weight Plastic Rubbing Strip

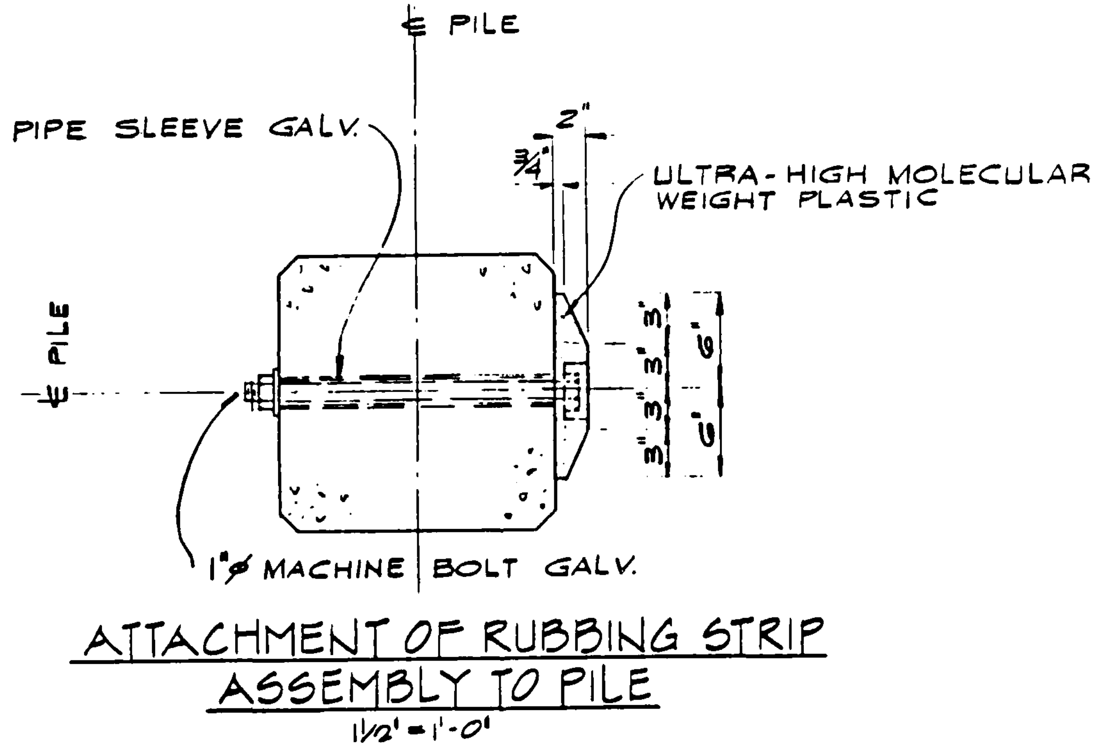


Figure 8.4. Fender System with Ultra High Molecular Weight Plastic Rubbing Strip

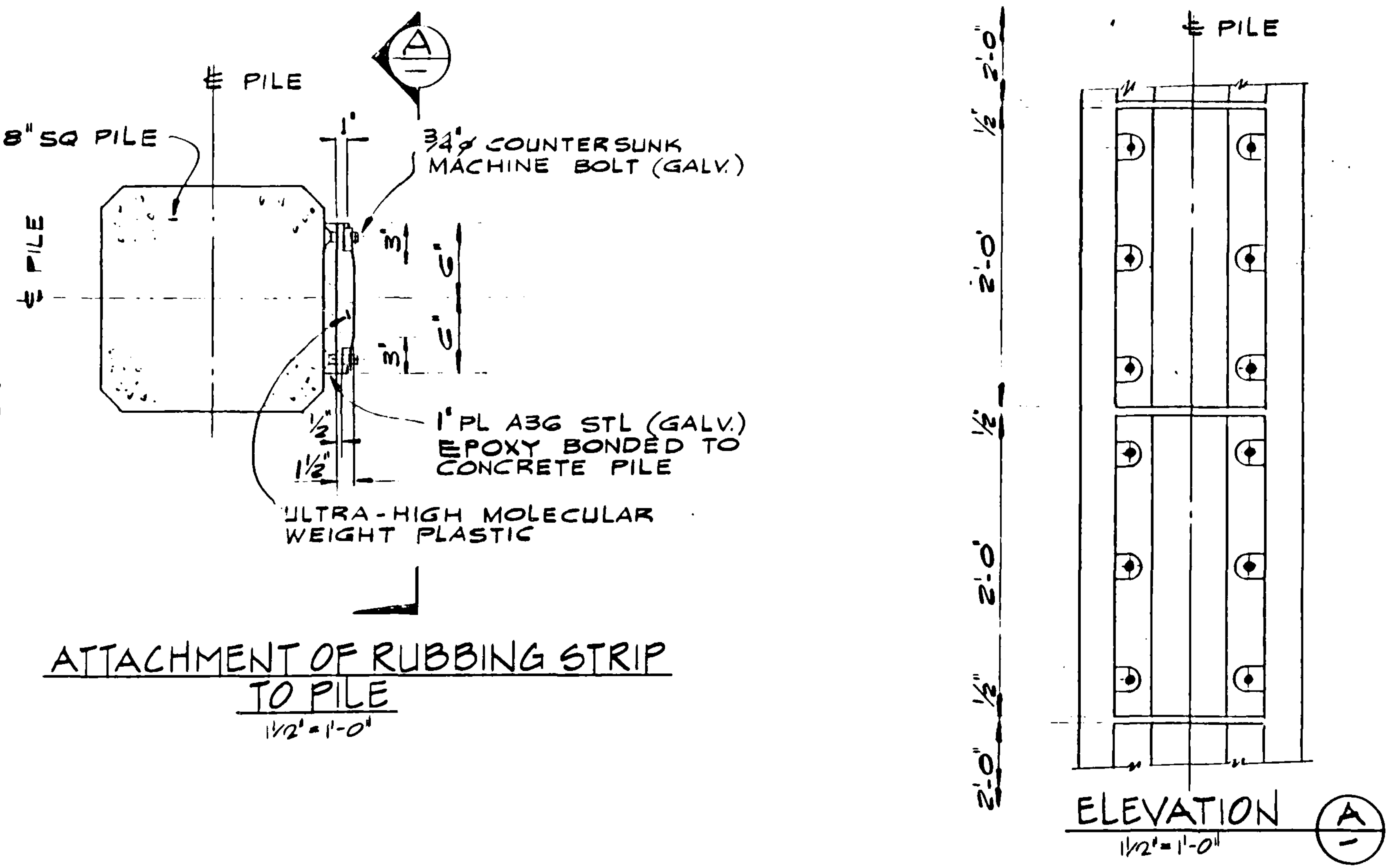


Figure 8.5 Fender System with Ultra High Molecular Weight Plastic Rubbing Strip

In Figure 8.3, the rubbing strip is bolted to a 1-in.-thick plate. This plate is bolted with 1-in.-diameter bolts through the center of the pile.

In Figure 8.4, the rubbing strip is 2 in. thick and is bolted directly to the pile with a 1-in.-diameter bolt.

In Figure 8.5, the rubbing strip is bolted to a 1-in.-thick plate with countersunk bolts. The plate is in turn bonded directly to the concrete pile with epoxy.

8.2 TREATED DOUGLAS FIR AND WALERS

Figure 8.6 shows the waler attachment and the rubbing strip. The waler distributes the berthing load to a horizontal "Delta rubber fender." The Delta rubber fenders offer higher energy-absorption capacity than cylindrical fenders.

The rubbing strip material consists of treated Douglas fir. The 6-in. x 12-in. rubbing strip has deep recesses for the attaching bolts. The rubbing strip is bolted to pile through the center of the pile.

8.3 RUBBER FENDERS

In Figure 8.7 the rubbing strip material consists of rubber. The rubbing strip is a wing-type fender which is bolted to the pile directly. The metal insert consists of stainless steel because it cannot be replaced.

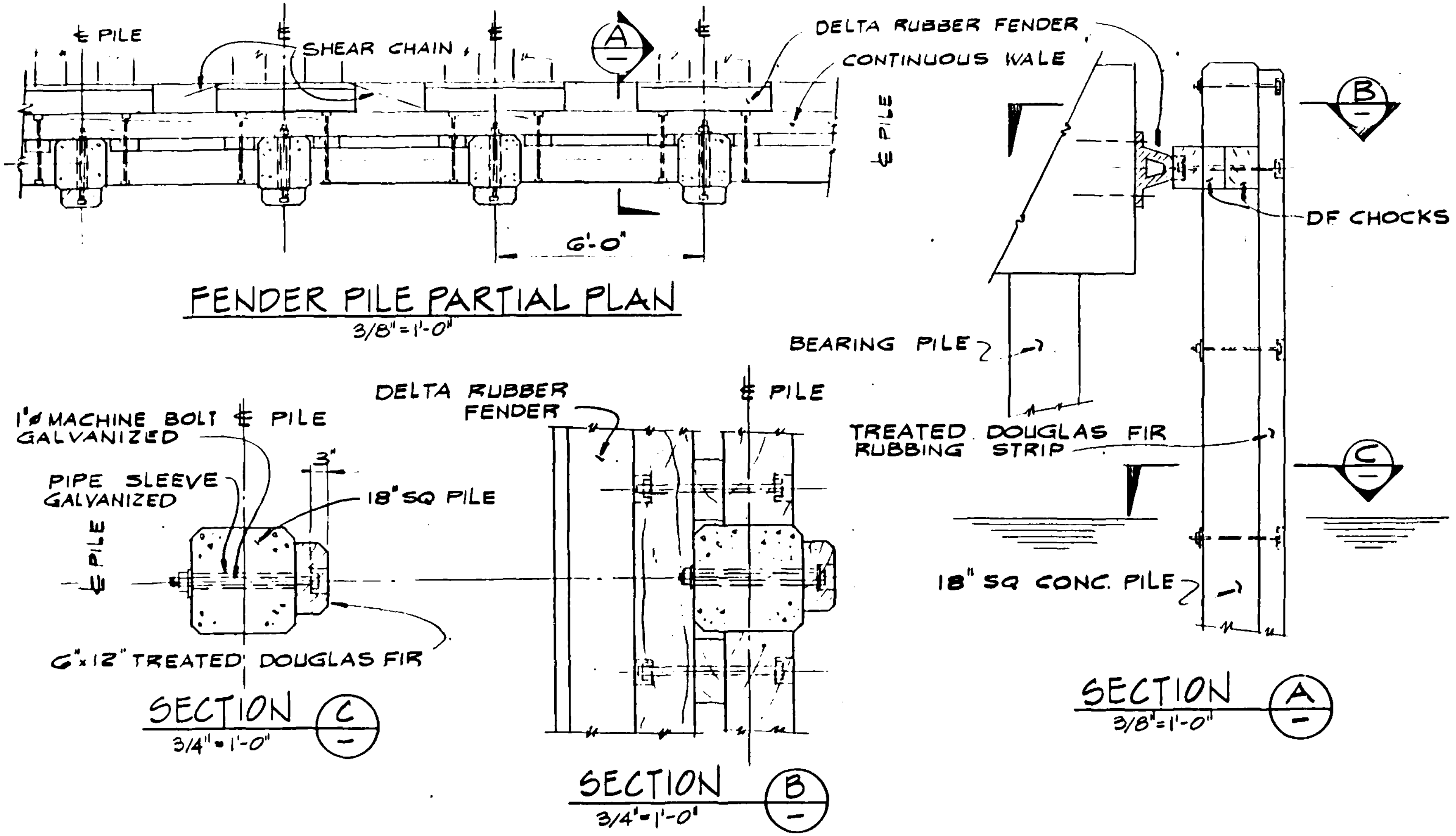
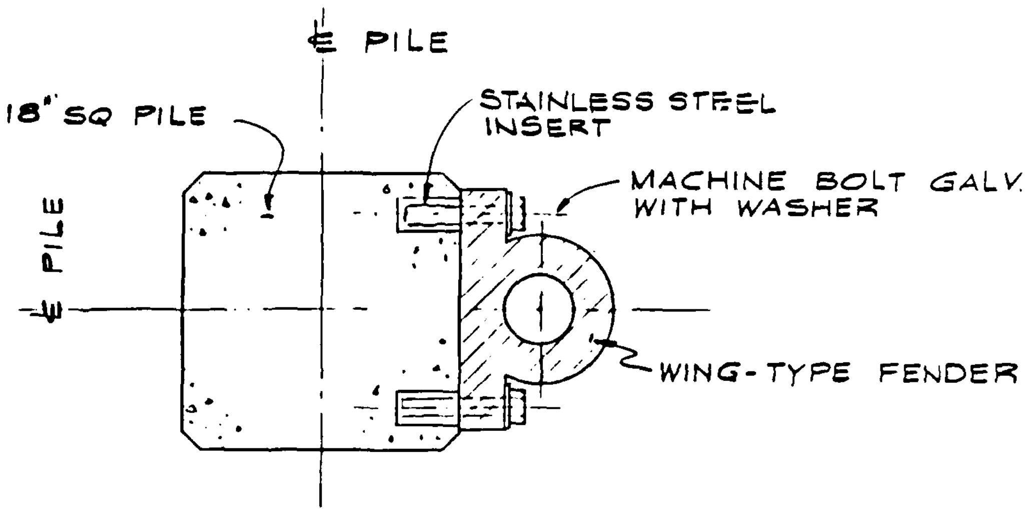


Figure 8.6. Fender System with Treated Douglas Fir Rubbing Strip



ATTACHMENT OF RUBBING STRIP TO PILE

1½" = 1'-0"

Figure 8.7. Fender System with Wing-Type Fender for Rubbing Strip

SECTION 9
CAPABILITIES OF PRESTRESSED CONCRETE PLANTS TO MANUFACTURE FENDER PILES

To determine the capabilities of existing prestressed concrete plants to manufacture prestressed concrete fender piles, a survey was taken of plants located on the three coasts of the continental United States and Hawaii. The survey was made by sending a questionnaire and pile drawing to approximately 30 companies. The survey form asked the companies to complete the two-page questionnaire and return it with any additional comments. Questions included in the form concerned the following subjects:

- Does the plant produce prestressed concrete piles, and in which sizes
- Maximum practical concrete strength
- Use of silica fume, HRWR admixtures, and steel fiber reinforcement
- Use of epoxy-coated prestressing strand

A summary of the results is presented in Table 9.1. A copy of the survey forms is included in Appendix D.

The results indicated that most of the plants are able to produce piles in the 12- to 18-in. size. The most common shape is square. Some plants are not able to produce 8000-psi concrete and have limited strength to 7000 psi. Three companies limited strength to 6000 psi. Most companies were familiar with silica fume concrete and HRWR admixtures (superplasticizers). Most companies either had little experience or preferred not to use epoxy-coated strand and steel fiber reinforcement.

Overall, there was much interest in the subject of prestressed concrete fender piles. The cover letter sending the survey form to the plants

TABLE 9.1
QUESTIONNAIRE SUMMARY

Region	Plant	Location City/State	Sizes Square (Future)	Sizes Octagonal	8,000 psi Comp. Str.(1)	Standard Max. Agg.	Silica Fume	HRWR	Epoxy Strand	Spiral Preferred	Strand Dia. 0.5 or 0.6 in	W11 Sprial	Partial Prest.	Mild Steel	Steel Fibers
W. Coast	JH Pomeroy	Petaluma/Ca	10,12,14,16,18,20,24	15,18	Yes	1"	Yes	Yes	Yes	Circular	Both	Yes	Yes	Yes	Yes
W. Coast	Rockwin	Santa Fe Springs/Ca	10,14,16,18,20	16,18,24	6,000	1/2"&3/4"	No	Yes	No	Circular	0.5"	Yes	Yes	Yes	No
W. Coast	Assoc. S&G	Everett/Washington	12	16,18,24	Yes	3/4"	Yes	Yes	Yes	Circular	0.5"	Yes	Yes	Yes	No
W. Coast	Kie-Con	Antioch/Ca	10-54		7,000	1"	No	No	No	Circular	Both	Yes	Yes	Yes	No
N. East	Blakeslee	Branford/Ct	Up to 24		Yes	1/2"&3/4"	Yes	Yes	Yes	Circular	Both	Yes	Yes	Yes	No
N. East	Lonestar	Littleton/Mass.	12,14,16		Yes	3/4"	No	Yes	No	Square	Both	W3.5	Yes	Yes	Yes
N. East	*Precast*	Auburn/Maine	15,18		Yes	1/2"&3/4"	Yes	Yes	No	Either	Both	Yes	Yes	Yes	No
N. East	Unistress	Pittsfield/Mass.	12,14, (16,18)		6,000	-	Yes	Yes	No	Circular	Both	1/4"	Yes	Yes	No
N. East	Formigli	Berlin/NJ	12-24		Yes	3/4"	Yes	Yes	Yes	Square	Both	W10	Yes	Yes	No
South	Gulf Coast	Pass Christian/MS	10,12,14,16,18,20		7,000	1"	Yes	Yes	Yes	Square	Both	1/4"	Yes	Yes	No
South	Pre. Stress	Charleston/S.C.	10,12,14,18,20		6,000	1"	No	No	Yes	Circular	Both	No	Yes	Yes	No
South	S. Pre. Con.(2)	Pensacola/Fl	10,12,20,18,24		7,000	3/4"	Yes	Yes	Yes	Square	1/2"&9/16"	Yes	Yes	Yes	No
South	Gate Concrete	Jacksonville/Fl	10-24,30,36		Yes	3/4"	Yes	Yes	Yes	Square	Both	Yes	Yes	Yes	No
Hawaii	Hawaiian Dred	Ewa Beach/Hawaii	12,20	16.5	7,000	3/4"	Yes	Yes	Yes	Circular	Both	Yes	Yes	Yes	No
W. Coast	C.T.C.	Tacoma/Washington	10,12,14	14,16,20,24	Yes	1/2"	Yes	Yes	No	Circular	Both	Yes	Yes	Yes	Yes
W. Coast	Eugene S&G	Eugene/Oregon	12	14,16	6,500	3/4"	No	No	Yes	Circular	Both	Yes	Yes	Yes	No
W. Coast	Morse Bro's	Clackamas/Oregon	12,18,20,24	14,16,18	Yes	3/4"	No	Yes	Yes	Circular	Both	Yes	Yes	Yes	No

(1) Yes = Capability to produce 8,000 psi concrete
(2) Southern Prestressed Concrete has four facilities on the Gulf Coast including: (a)Tallahassee, Fl.; (b) Montgomery, Al.; and (c) Huntsville, Al.

indicated results of the survey would be sent to those plants participating in the survey. Undoubtedly, this helped to raise interest in the program.

The impact of the survey revealed several important items for future consideration.

- o Concrete strength of 8000 psi may not be achievable in all parts of the country. It may be necessary to provide designs for 6000-, 7000-, and 8000-psi concrete strengths.

- o Alternate pile sizes should be allowed to be furnished by fabricators. This will make best use of existing forms.

- o Fabricators are not very experienced with fiber-reinforced concrete. The benefits of its use should be carefully considered before specifying it in the piles.

- o Most fabricators preferred circular spirals, although square spirals appear not to be a major concern.

SECTION 10
TESTING PROGRAM

The analytical techniques have been refined in this phase of the project to a level that closely approximates the results obtained in the preliminary testing program. The final phase of testing will be directed towards verification of the analytical modeling and development of a more serviceable prestressed concrete fender pile. The procedures developed by NCEL for the preliminary testing will be applicable to the testing recommended in this report. Piles are to be tested monotonically unless recommended otherwise. The development of our recommendations for procurement of test piles are based on the following considerations.

10.1 TEST PILE RECOMMENDATIONS

a. Baseline Piles

Prestressed concrete piles based on the optimum design developed to a specific energy level in this phase of the report will be tested to confirm the current analytical modeling techniques.

b. Baseline Piles with High-Level Cyclic Testing

The preliminary testing program established the cyclic behavior of prestressed concrete piles over a substantial portion of a prestressed concrete pile's total load range. The next phase of cyclic testing should be directed toward substantiating the optimum pile's behavior at its recommended design working energy of 23 kip-ft (for a 65-ft pile). The working loads should be applied at least 100 times to provide a reasonable assessment of the pile's ability to withstand cyclic loading.

Following the application of the working loads (assuming successful completion), loads closer to the ultimate capacity of the pile need to be applied and cycled to failure to determine high load, low cycle behavior of the pile. These loadings should be in the range of 95% of the calculated ultimate load. Applied loadings should be fully removed after each cycle. This should assess the performance of the pile for both working loads and impact loadings near to, yet slightly below the point of cover spalling.

c. Baseline Piles with Loads Closer to the Support

The prestressed piles have been designed with a low initial prestress in the strand. This has been done to utilize the large stress range available in the prestressing strands to absorb impact energy. However, the embedment length required for partially prestressed strands increases substantially as the initial prestressing level is decreased. This is required to account for the effects of Poisson's ratio (i.e., the diameter of the strand is decreased with an increase in stress) on the strands that are subjected to a large stress range. Piles need to be tested to determine embedment constraints of the strand as the load approaches the support.

d. Sawcut Compression Face

The spalling of the cover initiates large rotations (see paragraph g) which result in strand rupture at pile failure. Cutting grooves across the compression face of the pile may relieve the compressive strains at the surface of the pile, thereby forcing the compressive forces downward within the portion of the pile confined by the spiral reinforcing. Thus, when the concrete cover spalls, the large concentrated rotations due to the downward shift in the internal compressive force should not be as large since the compressive force has already

been forced downward by the cuts. In addition, the confined concrete should be able to withstand higher strains with a corresponding improvement in ductility. The cuts in the compression face will reduce the ultimate moment capacity of the pile, but this may be offset by an increase in the pile's ductility and energy-absorbing ability.

e. Mild Steel Reinforcing

Mild steel reinforcing has a high strain capacity after yielding has occurred as compared to prestressing strand; therefore, it may improve the failure characteristics of the pile following spalling of the cover at ultimate load. However, the mild steel reinforcing will yield in tension at a relatively low berthing load due to its yield capacity of 60 ksi as compared to the prestressing strand's yield capacity of 243 ksi for 270-ksi strand. The prestress in the pile will reclose the crack, thereby causing the reinforcing steel to yield again in compression. Also, the reinforcing steel may make the pile less serviceable at low energy levels by hindering crack closure upon removal of the berthing load. These test piles need to be cyclically loaded to determine their fatigue and serviceability characteristics before loading them to ultimate. Cycle these test piles incrementally in accordance with the procedures developed for the preliminary testing program.

f. Slack Strand Reinforcing

Slack strand may also improve the failure characteristics of the pile following spalling of the cover at ultimate load. Tensile yielding of the slack strand is not as important as for the mild steel reinforcing, but it is difficult to assess the effects of compression on the slack strand. Will the concrete confine the strand or will the strand induce premature failure of the concrete cover? To assess the performance of the slack

strand in tension and avoid the concern with compression, the test piles have been detailed with slack strand only on the tension face. In addition, providing slack strand on both faces would add substantially to the cost of the pile. Actual fender piles constructed with slack strands on one face would require extra care during driving operations to assure they are installed in the correct orientation. These piles also need to be cyclically loaded before loading them to ultimate. Cycle these test piles incrementally in accordance with the procedures developed for the preliminary testing program.

g. Fiber-Reinforced Concrete

Fiber-reinforced concrete may have a significant effect on the spalling characteristics of the concrete cover, with a corresponding increase in the performance of the prestressed pile beyond its ultimate moment capacity. However, the ultimate moment capacity of the prestressed fiber-reinforced concrete pile should not be much different than for a pile with ordinary high strength concrete. If the cover of a fiber-reinforced concrete pile does not spall at its ultimate moment capacity, the large local rotations observed on the preliminary test piles at the point of spalling will not occur. These large local rotations in the spall zone concentrated strains, in the prestressing strands under tension, to a relatively short length of the strand, which led to failure of the pile with minimum ductility. This concentration of strain could be prevented if the cover does not spall. Thus, the strains in the prestressing strand should be distributed more uniformly along the length of the pile, thereby better utilizing the strain characteristics of the strand. This should result in a pile being able to absorb substantially more energy beyond its ultimate moment capacity without the damage observed in the preliminary testing program.

h. Confinement Reinforcing

The confinement reinforcing for the test piles in the preliminary testing program was chosen to provide confinement of the concrete following spalling of the cover. This was not as effective as anticipated since the prestressing strands broke shortly after spalling due to the concentrated rotations and strains that occur at the point of spalling. The cross ties and longitudinal rebars used in the preliminary test program and in the majority of piles in the final test program need to be removed to assess their effect on the pile's capacity. In addition, the spacing of the spiral reinforcing needs to be increased to determine its effect on the capacity of the pile. Heavy spiral reinforcing, as currently used on the test piles, has a significant impact on the cost of a prestressed fender pile. Therefore, if the spacing or size of the spiral can be increased or reduced, respectively, the initial cost of a prestressed concrete fender pile will be reduced.

The above recommendations were consolidated into three reinforcement configurations. These pile configurations and the different alternatives in which they are used are shown on the pile drawing in Appendix A. A summary of the pile recommendations is given in Table 10.1. R/E curves for the reinforcement configurations are presented in Figure 10.1 These curves are based on 65-ft-long piles.

In addition, we recommend that corrosion testing be performed. Field experience and a literature review indicate that single transverse cracks in a prestressed concrete pile have a low potential for corrosion. However, the effects of multiple closely spaced cracks are largely unknown. It is recommended that NCEL consider the development of a testing program to assess the impact of long-term corrosion on prestressed concrete piles with multiple cracks. Test samples can be cut from portions of the test piles described above.

TABLE 10.1
SUMMARY OF TEST PILE RECOMMENDATIONS

Pile Description		Configuration	M-K Nos.	Loading: Monotonic	Loading: Cyclic	Comments
a.	Baseline Pile	1	9, 10	X		Confirm analysis
b.	Baseline Pile	1	11, 12		X	High-level cyclic load
c.	Baseline Pile	1	13, 14	X		Load close to support
d.	Baseline Pile	1	15	X		Saw cuts in compression face
e.	Mild Steel Reinforcement	2	16, 17		X	Load incrementation
f.	Slack Strand Reinforcement	3	18, 19		X	Load incrementation
g.	Baseline Pile	1	20, 21	X		Fiber-reinforced concrete
h.	Baseline Pile	1	22-24	X		Vary spiral pitch

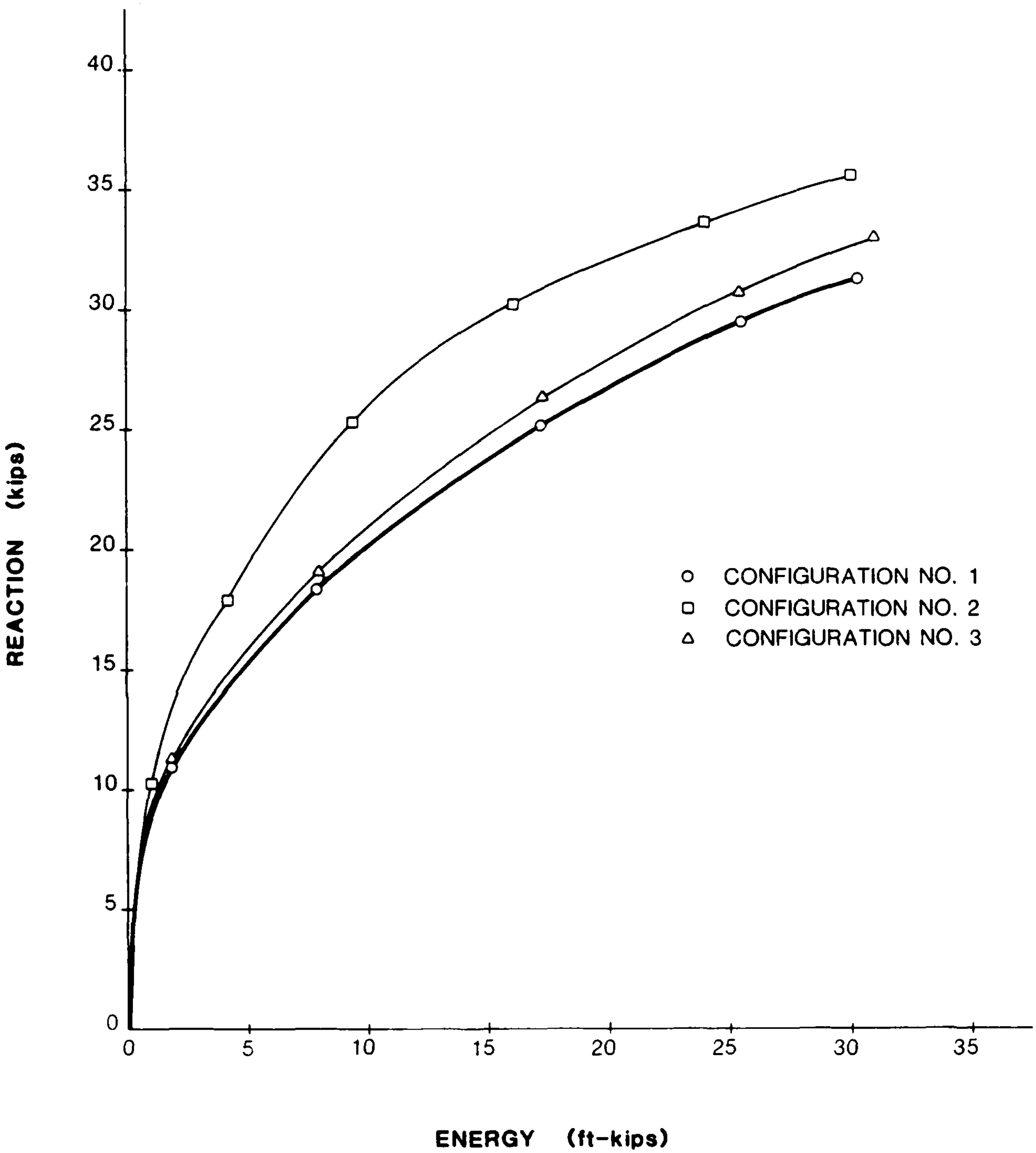


Figure 10.1. R versus E for Test Pile Configurations

SECTION 11
DRAWINGS AND SPECIFICATIONS

A drawing and set of specifications were prepared as a part of this phase of the development of prestressed concrete fender piles for procurement of test piles for the upcoming testing program. The test piles were developed to meet the testing program recommendations outlined in Section 10 of this report. The test pile drawing and specifications can be found in Appendix A and Appendix B, respectively.

REFERENCES

1.1 Naval Facilities Engineering Command. Test and Evaluation Master Plan: Development of Prestressed Concrete Fender Piles, Program Element 63725N, Project No. Y0995-SL. Alexandria, VA, June 1984.

2.1 Naval Civil Engineering Laboratory. Contract Report: Prestressed Concrete Fender Piles - Analysis and Test Pile Detailing. Federal Way, WA., ABAM Engineers Inc., 25 January 1985. (NCEL Contract No. N62474-84-C-3140).

2.2 Naval Civil Engineering Laboratory. Technical Memorandum: Development of Prestressed Concrete Fender Piles - Preliminary Tests.

3.1 American Concrete Institute (ACI), ACI 318-83, Building Code Requirements for Reinforced Concrete.

3.2 S.M. Morales. Short-term Mechanical Properties of High Strength Light- weight Concrete, PhD thesis, Department of Structural Engineering, Cornell University. Ithaca, NY, August 1982.

3.3 Juan A. Pastor. Behavior of High-Strength Concrete Beams, Department of Structural Engineering, Cornell University. Ithaca, NY, February 1984.

3.4 Prestressed Concrete Institute, PCI Design Handbook, 2nd edition.

5.1 American Association of State Highway and Transportation Officials (AASHTO), Standard Specifications for Highway Bridges, 1983.

5.2 Economic Analysis Handbook, NAVFAC P-442, July 1980.

6.1 Private Communication, Florida DOT Bridge Inspection Department, Summary of Condition Survey.

6.2 P. Kumar Mehta. Performance of Concrete in Marine Environment, Publication SP-65, American Concrete Institute.

6.3 P. Kumar Mehta and Ben C. Gerwick, Jr. "Cracking - Corrosion Interaction in Concrete Exposed to Marine Environment," Concrete International (ACI), October 1982.

6.4 A.W. Beeby. Cracking, Cover, and Corrosion of Reinforcement, Concrete International (ACI), February 1983.

6.5 ACI Committee 222 Report, Corrosion of Metals in Concrete, ACI Journal, January/February 1985.

6.6 Harvey H. Haynes. Permeability of Concrete in Sea Water, Performance of Concrete in Marine Environment, Publication SP-65, ACI.

6.7 National Cooperative Research Program (NCHRP), Report Number 244. Protective Coatings for Concrete.

6.8 Private Communication with David Stark, Portland Cement Association, Skokie, Illinois.

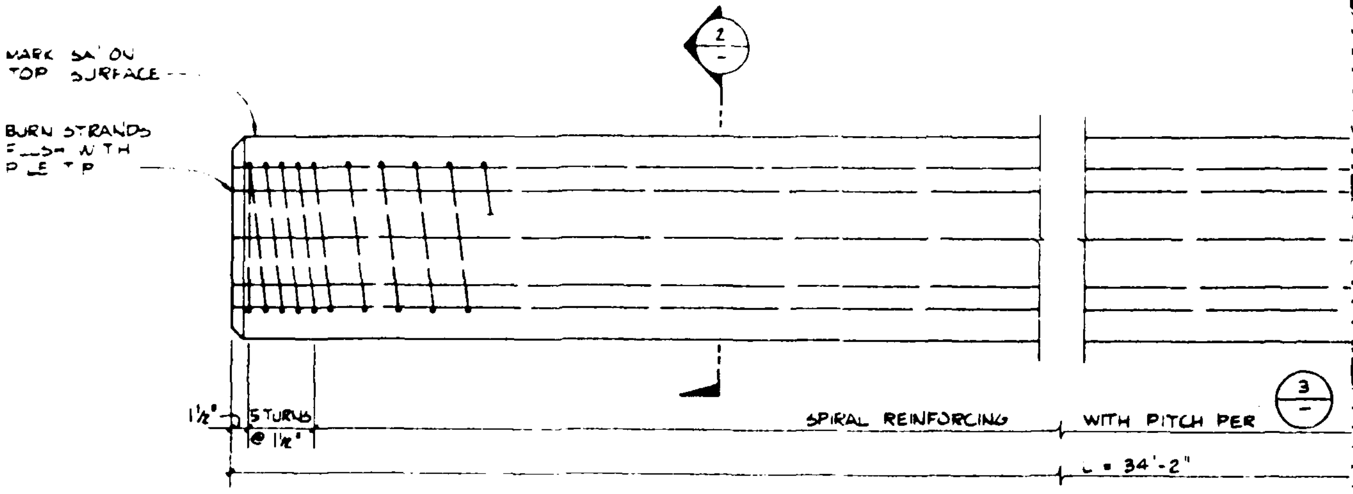
7.1 American Concrete Institute (ACI), ACI 544.1R, ACI Manual of Standard Practice, Part 5.

NOTATIONS

C	Neutral axis depth (in.)
C_z	Distance from neutral axis to bottom of rectangular stress block (in.)
E_c	Modulus of elasticity of concrete, psi
E_S	Modulus of elasticity of reinforcement, psi
E_u	Ultimate energy, the calculated energy at an ultimate concrete strain of 0.003 in./in., ft-kips
E_w	Working energy, ft-kips
f_b	Allowable bending stress, psi
f'_c	Average compressive strength of concrete, psi
f_{pc}	Compressive stress in concrete due to effective prestress force only (after allowance for all prestress losses), psi
f_{pu}	Specified tensile strength of prestressing tendons, psi
f_r	Modulus of rupture for timber piles, psi
f_{se}	Effective stress in prestressed reinforcement (after allowance for all prestress losses), ksi

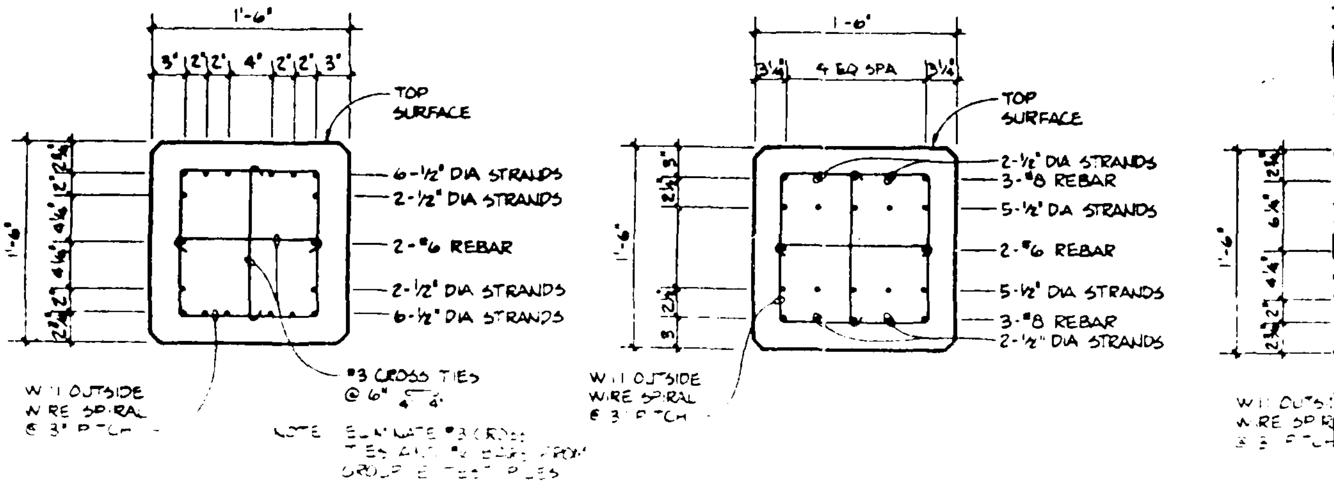
f_t	Tensile stress in concrete, psi
f_y	Specified yield strength of nonprestressed reinforcement, psi
M_n	Nominal moment strength of pile, kip-ft
R/E	Reaction to energy ratio calculated at ultimate energy
R_d	Design reaction, kips
R_u	Calculated reaction at ultimate energy, kips
Δ_d	Calculated deflection at design reaction, in.
Δ_u	Calculated deflection at ultimate energy, in.
Δ_E	% change from baseline
ϕ	Strength reduction factor
α	Maximum stress factor
ε_c	Concrete strain
ε_{pz}	Concrete strain at maximum compressive stress

APPENDIX A
DRAWINGS



1 — 18" PRESTRESSED CONCRETE PILE

SCALE : 1½" = 1'-0"



CONFIGURATION NO. 1

CONFIGURATION NO. 2

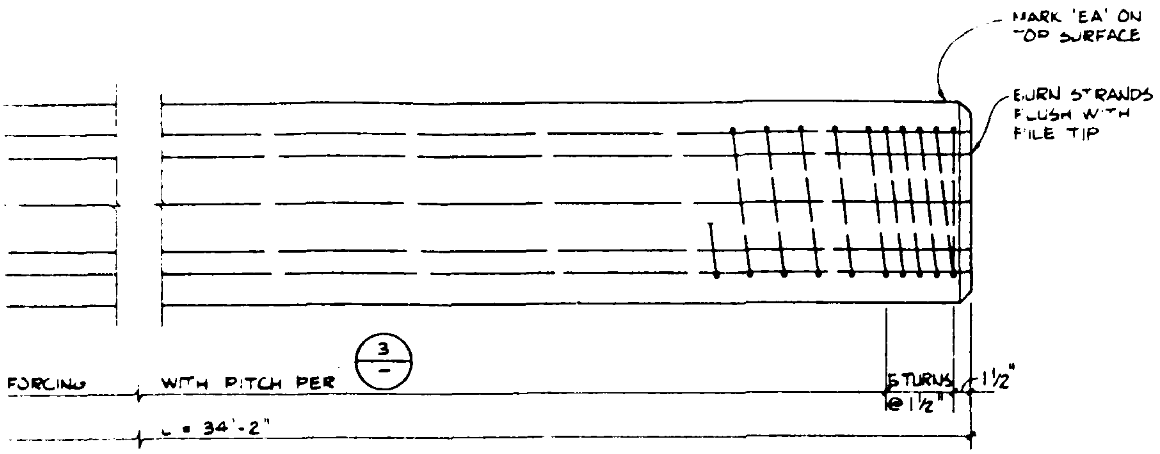
NOTE : DIMENSIONS ARE SHOWN TO CENTERLINE OF STRAND OR REBAR. MAINTAIN 2" MIN. CONCRETE COVER TO ALL STEEL

2 — PILE SECTIONS

SCALE : 1½" = 1'-0"

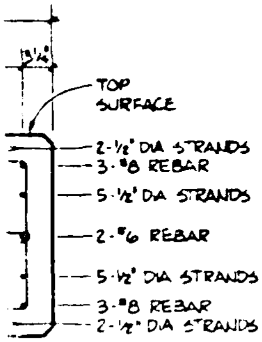
NOTES

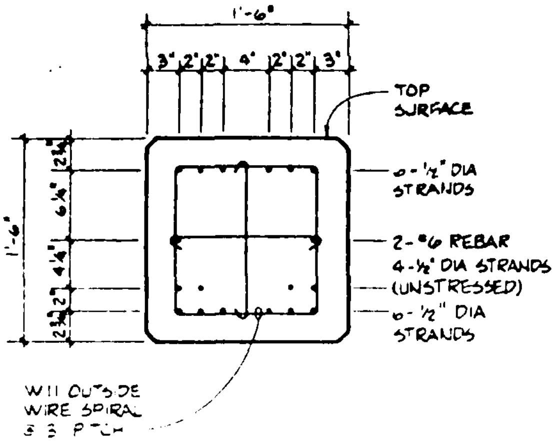
1. LIFTING DEVICE LOCATIONS A[...] DETERMINED BY THE FABRIC[...] CONSIDERING HANDLING STR[...] AND TESTING REQUIREMEN[...]
2. PRECAST CONCRETE PILES [...] f'_c = 8000 PSI AT 28 D[...]
3. REINFORCEMENT
   - A. REBAR . ASTM A615, [...]
   - B. PRESTRESSING STEEL[...] SEVEN WIRE, UNCOATE[...] RELAXATION STRANDS F[...]
   - C. SPIRAL REINFORCEME[...] DRAWN WIRE REINFORC[...] ASTM A82, Fy = 70 [...]
4. MARK EACH END OF PILE A[...]



RESSED CONCRETE PILE

1'-0"





N NO. 2

CONFIGURATION NO. 3

OWN TO CENTERLINE BAR, MAINTAIN 2" MIN. TO ALL STEEL

CTIONS

1'-0"

3/— PILE DATA

ITEM	TEST PILE GROUP			
	A	B	C	
CONFIGURATION	1	2	3	
SIZE	18" x 18"	18" x 18"	18" x 18"	18"
SECTION AREA	324 IN²	324 IN²	324 IN²	32
TENSIONING UNITS	16 STRANDS	14 STRANDS	12 STRANDS	16 ST
INITIAL PRESTRESS FORCE	164 k	162 k	160 k	16
DESIGN PRESTRESS FORCE	146 k	146 k	146 k	14
DESIGN CONCRETE STRESS	450 PSI	450 PSI	450 PSI	45
SPIRAL REINFORCEMENT	3" PITCH	3" PITCH	3" PITCH	3"
QUANTITY	7	2	2	
MARK NOS.	9 THRU 15	16 & 17	18 & 19	20
CONCRETE	NORMAL	NORMAL	NORMAL	FI REINF

NOTES: 1. PROVIDE 1½" DEEP & FULL WIDTH CUTS ON TOP SURF[...] OF MARK 15 @ 11'-10", 13'-4", 14'-10", 16'-4", 17'-10", 19'-4", 2[...] & 22'-4" (8 CUTS TOTAL) FROM END MARKED 'SA'.

2. MARK 22 - 3" SPIRAL PITCH W/O CROSS TIES & RE[...]
MARK 23 - 4½" SPIRAL PITCH W/O CROSS TIES & R[...]
MARK 24 - 6" SPIRAL PITCH W/O CROSS TIES & RE[...]

NOTES

1. LIFTING DEVICE LOCATIONS ARE TO BE DETERMINED BY THE FABRICATOR CONSIDERING HANDLING STRESSES AND TESTING REQUIREMENTS.
2. PRECAST CONCRETE PILES f'_c = 8000 PSI AT 28 DAYS
3. REINFORCEMENT
   - A. REBAR . ASTM A615, GR 60.
   - B. PRESTRESSING STEEL : 1/2" DIA, 270 KSI SEVEN WIRE, UNCOATED LOW RELAXATION STRANDS PER ASTM A416.
   - C. SPIRAL REINFORCEMENT : W11 COLD DRAWN WIRE REINFORCEMENT AS PER ASTM A82, Fy = 70 KSI,
4. MARK EACH END OF PILE AS SHOWN.

MARK 'EA' ON TOP SURFACE

BURN STRANDS FLUSH WITH PILE TOP

(3) PILE DATA

	TEST PILE GROUP				
	A	B	C	D	E
ION	1	2	3	1	1
	18" x 18"	18" x 18"	18" x 18"	18" x 18"	18" x 18"
REA	324 IN²	324 IN²	324 IN²	324 IN²	324 IN²
NTS	16 STRANDS	14 STRANDS	12 STRANDS	16 STRANDS	16 STRANDS
TRESS FORCE	164 K	162 K	160 K	164 K	164 K
TRESS FORCE	146 K	146 K	146 K	146 K	146 K
RETE STRESS	450 PSI	450 PSI	450 PSI	450 PSI	450 PSI
ORCEMENT	3" PITCH	3" PITCH	3" PITCH	3" PITCH	VARIES
	7	2	2	2	3
	9 THRU 15	16 & 17	18 & 19	20 & 21	22 THRU 24
	NORMAL	NORMAL	NORMAL	FIBER REINFORCED	NORMAL

ROVIDE 1/2" DEEP x FULL WIDTH CUTS ON TOP SURFACE F MARK 15 @ 11'-10", 13'-4", 14'-10", 16'-4", 17'-10", 19'-4", 20'-10", 22'-4" (8 CUTS TOTAL) FROM END MARKED '5A'.

ARK 22 - 3" SPIRAL PITCH W/O CROSS TIES & REMOVE #6 BARS
ARK 23 - 4 1/2" SPIRAL PITCH W/O CROSS TIES & REMOVE #6 BARS
ARK 24 - 6" SPIRAL PITCH W/O CROSS TIES & REMOVE #6 BARS

ABAM
CONSULTING ENGINEERS
500 SOUTH 336TH STREET FEDERAL WAY WA 98003

Sheet Title: PILES – PHASE IV

Project Title: NCEL – PRESTRESSED CONCRETE FENDER PILES CONTRACT N62474-84-C-3140

File No: A85003
Date: 24 JAN 86
Designer: CWS
Drafter: KRW
Project Engr: MHZ
Project Mgr: JRS
Drawing No
Sheet of

APPENDIX B
SPECIFICATIONS

SPECIFICATIONS

NAVAL CIVIL ENGINEERING LABORATORY (NCEL)

EIGHTEEN-INCH PRESTRESSED CONCRETE TEST PILES

PART 1 - GENERAL

1.01 PURPOSE

A. Cast 16 concrete prestressed piles, as per Test Pile Groups A, B, C, D, and E; ABAM design drawing dated 24 January 1986.

1.02 RELATED WORK - SPECIFIED HEREIN

A. Fabrication of all reinforcement and prestressing steel for each pile group as specified.

B. Concrete mix design for normal weight concrete containing 3 to 6 percent air entrainment.

C. Concrete mix design for normal weight concrete containing steel fibers and 3 to 6 percent air entrainment.

D. Cast and cure concrete piles for mark numbers as specified under pile data on the drawing.

E. Cocrete saw cutting and joint filling with Visco-elastic material.

F. Concrete and steel test specimens representing each pile.

G. Delivery to the NCEL Laboratory in Port Hueneme, California.

1.03 QUALITY ASSURANCE

Reference specifications and standards

A. ACI: 211.1-81 Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete

B. ACI: 318-83 Building Code Requirements for Reinforced Concrete

C. ACI: 347-78 Recommended Practice for Concrete Formwork

D. ACI: 544.1R-82 State-of-the-Art Report on Fiber Reinforced Concrete

E. ASTM: A 82-79 Cold Drawn Steel Wire for Concrete Reinforcement

F. ASTM: A 416-80 Uncoated Seven Wire Stress Relieved Strand for Prestessed Concrete

G. ASTM: A 615-82 Deformed and Plain Billet-Steel Bars for Concrete Reinforcement

H. ASTM: C 31-83 Making and Curing Concrete Test Specimens in the Field

I. ASTM: C 33-82 Concrete Aggregates

J. ASTM: C-150-83a Specification for Portland Cement

K. ASTM: C260-77 Specification for Air-Entraining Admixtures for Concrete

L. ASTM: C 469-83 Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression.

M. ASTM: C494-82 Specification for Chemical Admixtures for Concrete

N. ASTM: C618-83 Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete

O. PCI: Manual for Quality Control for Plants and Production of Precast Prestressed Concrete Products, PCI Publication Number MNL-116-77

PART 2 - MATERIALS

2.01 PORTLAND CEMENT

A. The tricalcium aluminate content (C_3A) shall be 6 to 10 percent. Alkalies as Na_2O shall be less than 0.60. Note: This reflects higher C_3A desired for corrosion resistance.

B. Manufacturer's certificate of compliance and a copy of a current mill test shall accompany mix design submittal.

2.02 MINERAL ADMIXTURES

Mineral admixtures, fly ash (Class F) and silica fume may be accepted on the basis of mill tests and manufacturer's certificate of compliance with ASTM C618.

2.03 ADMIXTURES

A. All concrete admixtures shall be approved by ABAM. No calcium chloride will be allowed nor admixtures containing chloride ions.

1. Air entraining: Per ASTM C260

2. Water-reducing admixtures: Per ASTM C494, Type A or D

3. High-range water reducing admixtures shall conform to ASTM C494 Type F or G

B. Preconstruction testing shall be conducted to document the fabricator's ability to produce workable concrete which does not segregate or show excessive slump loss characteristics.

2.04 AGGREGATES

Aggregates shall conform to ASTM C33 for quality and gradation in size Nos. 7 or 67. Maximum coarse aggregate size to be 3/4-in. Copies of current gradation and statements of compliance shall accompany mix submittal.

2.05 STEEL REINFORCEMENT

Reinforcement shall conform to requirements of PCI Manual MNL-116, Division III, Section 1.

A. Reinforcement bars: ASTM A615, Grade 60

B. Pretensioning: Uncoated, seven-wire, stress-relieved strand conforming to ASTM A416, Grade 270, low relaxation

C. Spiral reinforcement: Cold drawn wire reinforcement conforming to ASTM A82, Fy = 70 ksi

2.06 STEEL FIBERS

Steel fibers shall be Dramix ZP50/0.50, Ribtec Xorex 1, Ribtec Xorex II, or an approved equal. Fibers shall be 2 in. long.

2.07 VISCO-ELASTIC JOINT FILLER MATERIAL

Joint filler material shall be Thiokol or an approved equal.

PART 3 - CONSTRUCTION REQUIREMENTS

3.01 CONCRETE PROPORTIONING

A. General

Concrete shall be a uniform mix, capable of being placed without segregation. Fresh and hardened properties shall conform to requirements by specification herein.

B. Strength

The 28-day compressive strength, f'_c, shall be 8000 psi. The compressive strength at the time of pile testing is intended to be between 7500 and 8500 psi. Historical data of mixes shall be submitted prior to production of piling showing accurate predictions of strength at ages up to 90 days.

C. Proportioning

1. The water/cement ratio (excluding any mineral admixture) for each concrete mix shall be equal to or less than 0.40.

2. Concrete mix proportioning shall be in conformance to ACI 211.1. Documentation of compressive strength shall comply with ACI 318, Part 3, Section 4.3 - Proportioning on the Basis of Field Experience and/or Trial Mixtures.

3. Mineral admixtures shall be used. Silica fume from any approved source shall be added in the amount of 10 percent by weight of cement or Class F fly ash in the amount of 15 to 25 percent by weight of cement depending on cement and fly ash properties.

4. Air entrainment shall be in the range of 3 to 6 percent as measured in the fresh concrete.

3.02 STEEL FIBERS

Concrete with steel fibers shall be proportioned, mixed, and placed in conformance with the recommendations in ACI 544.1R. The fabricator shall prepare trial mixes for selection of final fiber content and concrete constituents to provide workability, ease of placing, and ease of finishing. The fiber content for the steel fibers shall be between 0.75 and 1.5 percent by volume of the concrete. The smaller amount shall be used if deformed wires are incorporated and the larger amount percentage shall be used if smooth wires are selected.

3.03 CONCRETE MIXING

General requirements for equipment, materials storage and handling, batching, placing, consolidation, and finishing shall be as defined in the PCI Manual MNL 116, Division II, Section 2.

3.04 FORMWORK

Formwork construction shall be in conformance to ACI Standard 347 and PCI Manual MNL 116.

3.05 CONCRETE PLACING

Concrete test piles shall only be fabricated from single batches of concrete (i.e., fabricating any single test pile from two different concrete batches will not be allowed).

3.06 CONCRETE CURING

A. Curing of all concrete shall be in accordance with PCI Manual MNL 116, Division I, Section 4.

B. An accelerated curing cycle to produce concrete compressive strength of f'_{ci} = 4500 psi in 24 hrs or less shall be used.

C. Concrete thermal protection shall be maintained after steam curing is completed until the difference between the concrete temperature and the ambient air temperature outside of the steam hoods is less than 30°F.

3.07 PRETENSIONING

A. PCI Manual MNL-116, Division I, Section 2.

B. Initial prestressing forces for each pile are listed under pile data on the drawing.

C. Strands shall be adequately supported at one-third points of the pile to prevent sagging of the strand.

3.08 DETENSIONING

PCI Manual MNL 116, Division I, Section 3

3.09 LIFTING DEVICES

The fabricator is responsible for the selection and placement of lifting devices to provide for proper handling and transportation as per PCI Manual MNL 116, Division IV, Sections 1 and 2.

3.10 CONCRETE SAW CUTTING

Concrete saw cutting shall be done a minimum of 10 days after casting but prior to shipping. Care shall be taken to ensure that reinforcing is not cut.

3.11 JOINT FILLER

Joint filler shall be installed in clean, dry saw cuts prior to shipping.

PART 4 - TESTING

4.01 CONCRETE

A. Concrete shall be tested in accordance with PCI Manual MNL 116, Division V, Section 3.

B. A minimum of 10 concrete test cylinders as per ASTM C31, shall be made for each of the 16 concrete piles (total of 160 test cylinders). Cylinders shall be cured at the same time and temperature as the concrete piles.

   1. Three cylinders per pile shall be tested for compressive strength vs. strain as per ASTM C 469 immediately prior to detensioning. Testing records shall be furnished to ABAM.

   2. Seven cylinders per pile are to be properly identified and sealed in moisture-retentive plastic bags and stored at the same ambient temperature as the concrete piles. They are to be packaged to prevent damage to the specimen and plastic bags and shipped in accordance with the same schedule as their respective test piling.

4.02 STEEL

A. A manufacturer's certificate of ASTM compliance is to be furnished for all steel reinforcement embedments.

B. Supply test specimens of 18-in. lengths of reinforcement bars, spirals, and strands. Supply three samples representing each type of reinforcement used in the piling.

C. Properly identify the test specimens for production traceability. Package and ship with the piling.

D. Provide manufacturer's certificates for the steel fibers showing tensile strength, Young's modulus, and ultimate elongation.

PART 5 - QUALITY CONTROL

5.01 FABRICATORS

A. Plant quality assurance and quality control procedures to comply with standards set forth by PCI MNL Manual 116 and the PCI Plant Certification Program.

B. ABAM to have option for final selection and approval.

5.02 PERSONNEL

Competent design, production, and testing personnel as per PCI Manual MNL 116.

PART 6 - DELIVERY

The 16 concrete test piles, companion concrete test cylinders, and steel test specimens shall be delivered FOB to the NCEL testing laboratories at Port Hueneme, California. Unloading of the piles and test specimens will be the responsibility of others.

DISTRIBUTION LIST

DTIC Alexandria, VA
GIDEP OIC, Corona, CA
NAS SCE, Alameda, CA
NAVFACENGCOM Code 03, Alexandria, VA; Code 04, Alexandria, VA; Code 04T4, Alexandria, VA; Code 10, Alexandria, VA
NAVFACENGCOM - CHES DIV Code FPO-1PL, Washington, DC
NAVFACENGCOM - LANT DIV. Code 04, Norfolk, VA; Library, Norfolk, VA
NAVFACENGCOM - NORTH DIV. Code 04, Philadelphia, PA; Code 04AL, Philadelphia, PA
NAVFACENGCOM - PAC DIV. Code 04, Pearl Harbor, HI; Code 402, RDT&E, Pearl Harbor, HI; Library, Pearl Harbor, HI
NAVFACENGCOM - SOUTH DIV. Code 04, Charleston, SC; Library, Charleston, SC
NAVFACENGCOM - WEST DIV. Code 04, San Bruno, CA; Code 04D1, San Bruno, CA; Code 402, San Bruno, CA, Library (Code 04A2.2), San Bruno, CA
NAVSTA SCE, Charleston, SC; SCE, Guam, Marianas Islands; SCE, Norfolk, VA; SCE, Pearl Harbor HI; SCE, San Diego CA; SCE, Treasure Is, San Francisco, CA
PWC CO, Guam, Mariana Islands; CO, Norfolk, VA; CO, Oakland, CA; CO, Pearl Harbor, HI; Code 101 (Library), Oakland, CA; Code 110, San Diego, CA; Code 123-C, San Diego, CA; Code 420, Great Lakes, IL; Library (Code 134), Pearl Harbor, HI; Library, Guam, Mariana Islands; Library, Norfolk, VA; Library, Pensacola, FL; Library, Yokosuka JA; Tech Library, Subic Bay, RP

END

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